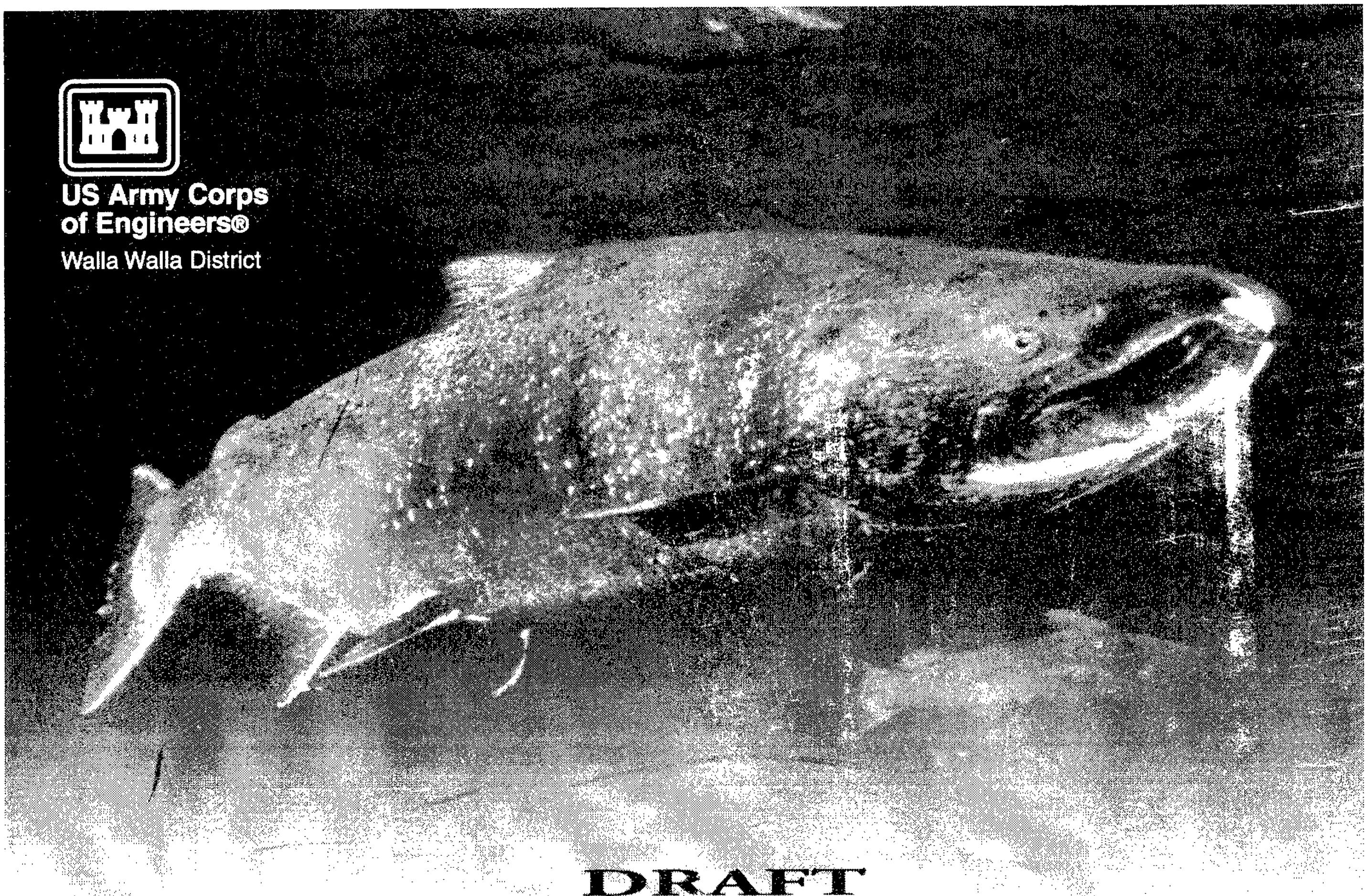

DRAFT

Lower Snake River Juvenile Salmon Migration Feasibility Report/ Environmental Impact Statement

APPENDIX D
Natural River Drawdown Engineering

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December 1999

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13. ABSTRACT *(Maximum 200 words)*

The Corps of Engineers along with the Bonneville Power Administration, US Environmental Protection Agency, and US Bureau of Reclamation as cooperating agencies, analyzed four general alternatives intended to provide information on the technical, environmental, and economic effects of actions related to improving juvenile salmon passage. The four alternatives include Alternative 1 - Existing Conditions (the no-action alternative) and three different ways to further improve juvenile salmon passage. The action alternatives are: Alternative 2 - Maximum Transport of Juvenile Salmon, Alternative 3 - Major System Improvements, and Alternative 4 - Dam Breaching. Comparison of the alternatives by all of the factors assessed in the study has not offered a clear-cut recommendation at this time. It is the Corps of Engineer's intent to recommend a preferred plan of action in the Final FR/EIS.

14. SUBJECT TERMS
Lower Snake River Project
Endangered Species Act
Fish Passage

15. NUMBER OF PAGES

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FEASIBILITY STUDY DOCUMENTATION

Document Title	
Summary to the Lower Snake River Juvenile Salmon Migration Feasibility Report/Environmental Impact Statement	
Lower Snake River Juvenile Salmon Migration Feasibility Report/Environmental Impact Statement	
Appendix A	Anadromous Fish
Appendix B	Resident Fish
Appendix C	Water Quality
Appendix D	Natural River Drawdown Engineering
Appendix E	Existing Systems and Major System Improvements Engineering
Appendix F	Hydrology/Hydraulics and Sedimentation
Appendix G	Hydroregulations
Appendix H	Fluvial Geomorphology
Appendix I	Economics
Appendix J	Plan Formulation
Appendix K	Real Estate
Appendix L	Lower Snake River Mitigation History and Status
Appendix M	Fish and Wildlife Coordination Act Report
Appendix N	Cultural Resources
Appendix O	Public Outreach Program
Appendix P	Air Quality
Appendix Q	Tribal Consultation/Coordination
Appendix R	Historical Perspectives
Appendix S	Snake River Maps
Appendix T	Biological Assessment
Appendix U	Clean Water Act, Section 404(b)(1) Evaluation

The documents listed above, as well as supporting technical reports and other study information, are available on our website at www.nww.usace.army.mil. Copies of these documents are also available for public review at various city, county, and regional libraries.

FOREWORD

This appendix is one part of the overall effort of the U.S. Army Corps of Engineers (Corps) to prepare the Lower Snake River Juvenile Salmon Migration Feasibility Report/Environmental Impact Statement (FR/EIS).

Please note that this document is a DRAFT appendix and is subject to change and/or revision based on information received through comments, hearings, workshops, etc. After the comment period ends and hearings conclude a Final FR/EIS with Appendices is planned.

The Corps has reached out to regional stakeholders (Federal agencies, tribes, states, local governmental entities, organizations, and individuals) during the development of the FR/EIS and appendices. This effort resulted in many of these regional stakeholders providing input, comments, and even drafting work products or portions of these documents. This regional input provided the Corps with an insight and perspective not found in previous processes. A great deal of this information was subsequently included in the Draft FR/EIS and Appendices, therefore, not all the opinions and/or findings herein may reflect the official policy or position of the Corps.

STUDY OVERVIEW

Purpose and Need

Between 1991 and 1997, due to declines in abundance, the National Marine Fisheries Service (NMFS) made the following listings of Snake River salmon or steelhead under the Endangered Species Act (ESA) as amended:

- sockeye salmon (listed as endangered in 1991)
- spring/summer chinook salmon (listed as threatened in 1992)
- fall chinook salmon (listed as threatened in 1992)
- steelhead (listed as threatened in 1997)

In 1995, NMFS issued a Biological Opinion on operations of the Federal Columbia River Power System. The Biological Opinion established measures to halt and reverse the declines of these listed species. This created the need to evaluate the feasibility, design, and engineering work for these measures.

The U.S. Army Corps of Engineers (Corps) implemented a study after NMFS's Biological Opinion in 1995 of alternatives associated with lower Snake River dams and reservoirs. This study was named the Lower Snake River Juvenile Salmon Migration Feasibility Study (Feasibility Study). The specific purpose and need of the Feasibility Study is to evaluate and screen structural alternatives that may increase survival of juvenile anadromous fish through the Lower Snake River Project (which includes the four lowermost dams operated by the Corps on the Snake River—Ice Harbor, Lower Monumental, Little Goose, and Lower Granite dams) and assist in their recovery.

Development of Alternatives

The Corps completed an interim report on the Feasibility Study in December 1996. The report evaluated the feasibility of drawdown to natural river levels, spillway crest, and other improvements to existing fish passage facilities. Based in part on a screening of actions conducted in the interim report, the study now focuses on four courses of action:

- Existing conditions (currently planned fish programs)
- System improvements with maximum collection and transport of juveniles (without major system improvements such as surface bypass collectors)
- System improvements with maximum collection and transport of juveniles (with major system improvements such as surface bypass collectors)
- Dam breaching or permanent drawdown to natural river levels for all reservoirs

The results of these evaluations are presented in the combined Feasibility Report (FR) and Environmental Impact Statement (EIS). The FR/EIS provides the support for recommendations that will be made regarding decisions on future actions on the Lower Snake River Project for passage of juvenile salmonids. This appendix is a part of the FR/EIS.

Geographic Scope

The geographic area covered by the FR/EIS generally encompasses the 140-mile long lower Snake River reach between Lewiston, Idaho and the Tri-Cities in Washington. The study area does slightly vary by resource area in the FR/EIS because the affected resources have widely varying spatial characteristics throughout the lower Snake River system. For example, socioeconomic effects of a permanent drawdown could be felt throughout the whole Columbia River Basin region with the most effects taking place in the counties of southwest Washington. In contrast, effects on vegetation along the reservoirs would be confined to much smaller areas.

Identification of Alternatives

Since 1995, numerous alternatives have been identified and evaluated. Over time, the alternatives have been assigned numbers and letters that serve as unique identifiers. However, different study groups have sometimes used slightly different numbering or lettering schemes and this has lead to some confusion when viewing all the work products prepared during this long period. The primary alternatives that are carried forward in the FR/EIS currently involve four major alternatives that were derived out of three major pathways. The four alternatives are:

Alternative Name	PATH[1/] Number	Corps Number	FR/EIS Number
Existing Conditions	A-1	A-1	1
Maximum Transport of Juvenile Salmon	A-2	A-2a	2
Major System Improvements	A-2'	A-2c	3
Dam Breaching	A-3	A-3a	4

[1/] Plan for Analyzing and Testing Hypotheses

Summary of Alternatives

The **Existing Conditions Alternative** consists of continuing the fish passage facilities and project operations that were in place or under development at the time this Feasibility Study was initiated. The existing programs and plans underway would continue. Project operations, including all ancillary facilities such as fish hatcheries and Habitat Management Units (HMUs) under the Lower Snake River Fish and Wildlife Compensation Plan (Comp Plan), recreation facilities, power generation, navigation, and irrigation would remain the same unless modified through future actions. Adult and juvenile fish passage facilities would continue to operate.

The **Maximum Transport of Juvenile Salmon Alternative** would include all of the existing or planned structural and operational configurations from the Existing Conditions Alternative. However, this alternative assumes that the juvenile fishway systems would be operated to maximize fish transport from Lower Granite, Little Goose, and Lower Monumental and that voluntary spill would not be used to bypass fish through the spillways (except at Ice Harbor). To accommodate this maximization of transport some measures would be taken to upgrade and improve fish handling facilities.

The **Major System Improvements Alternative** would provide additional improvements to what is considered under the Existing Conditions Alternative. These improvements would be focused on using surface bypass collection (SBC) facilities in conjunction with extended submersible bar screens (ESBS) and a behavioral guidance system (BGS). The intent of these facilities is to provide more effective diversion of juvenile fish away from the turbines. Under this alternative the number of fish collected and delivered to upgraded transportation facilities would be maximized at Lower Granite, the most upstream dam, where up to 90 percent of the fish would be collected and transported.

The **Dam Breaching Alternative** has been referred to as the "Drawdown Alternative" in many of the study groups since late 1996 and the resulting FR/EIS reports. These two terms essentially refer to the same set of actions. Because the term drawdown can refer to many types of drawdown, the term dam breaching was created to describe the action behind the alternative. The Dam Breaching Alternative would involve significant structural modifications at the four lower Snake River dams allowing the reservoirs to be drained and resulting in a free-flowing river that would remain unimpounded. Dam breaching would involve removing the earthen embankment sections of the four dams and then developing a channel around the powerhouses, spillways, and navigation locks. With dam breaching, the navigation locks would no longer be operational, and navigation for large commercial vessels would be eliminated. Some recreation facilities would close while others would be modified and new facilities could be built in the future. The operation and maintenance of fish hatcheries and Habitat Management Units (HMUs) would also change although the extent of change would probably be small and is not known at this time. Project development, design, and construction span a period of nine years. The first three to four years concentrate on the engineering and design processes. The embankments of the four dams are breached during two construction seasons at year 4-5 in the process. Construction work dealing with mitigation and restoration of various facilities adjacent to the reservoirs follows dam breaching for three to four years.

Authority

The four Corps dams of the lower Snake River were constructed and are operated and maintained under laws that may be grouped into three categories: 1) laws initially authorizing construction of the project, 2) laws specific to the project passed subsequent to construction, and 3) laws that generally apply to all Corps reservoirs.

BOUNDARIES

State

County

KM
300 100 0

MILES
7.5 15 30

125,000 ACRES

GEOGRAPHIC INFORMATION SYSTEMS
Corps of Engineers
Walla Walla District

1 : 1,900,800

DRAFT

Lower Snake River

Juvenile Salmon Migration Feasibility Study

REGIONAL BASE MAP

1999

GIS Analysts & Graphic Assembly: Applied Technology Team (CENWW-PM-PD-E)
g:\regional\reg\plates\lsr\jsmeis\lsrg\lsrgbmgm.dgn:GISFILE PLOTTED: 25 OCT 1999

ABSTRACT

Technical Appendix D, Natural River Drawdown Engineering, was prepared by engineers and staff of the Walla Walla District, Corps of Engineers. Other contributors include Raytheon Engineers and Constructors, American Hydro, Corps Waterways Experiment Station, Voest-Alpine, the Corps of Engineers Hydroelectric Design Center, Thomas, Dean and Hoskins, Montgomery-Watson Engineers, CH2M Hill Engineers, and Project Time and Cost.

It summarizes the design and construction processes necessary to prepare for and implement the drawdown pathway so that the feasibility study impacts and costs are based on a reasonable implementation method and realistic implementation costs.

The study process was a collaborative effort of professionals active in the many disciplines affected by possible implementation activities. The appendix addresses the five main categories of drawdown implementation:

1) The water in each reservoir must be evacuated.

2) A portion of each dam must be removed to allow the entire river to flow freely.

3) A means to channelize the river is necessary to control the river as it flows around the abandoned structures.

4) The abandoned structures must be decommissioned and each facility secured against public access.

5) Numerous structures and facilities in the reservoirs must be modified in order to operate with lower water surface elevations.

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Walla Walla District

Draft

Lower Snake River Juvenile Salmon Migration Feasibility Report/ Environmental Impact Statement

Appendix D

Natural River Drawdown Engineering

Produced by
U.S. Army Corps of Engineers
Walla Walla District

Completed November 1999
Revised and released for review
with Draft FR/EIS
December 1999

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ACRONYMS AND ABBREVIATIONS

CCP	concrete cylinder pipe
CFR	Code of Federal Regulations
Corps	U.S. Army Corps of Engineers
CWCCIS	Civil Works Construction Cost Index System
Ecology	Washington State Department of Ecology
ESA	Endangered Species Act
Feasibility Study	Lower Snake River Juvenile Migration Feasibility Study
HMU	Habitat Management Unit
MCACES™	Micro Computer-Aided Cost Engineering System
NEPA	National Environmental Policy Act
NMFS	National Marine Fisheries Service
PCB	polychlorinated biphenyl
PG&E	Pacific Gas and Electric
RR	Railroad
SCS	System Configuration Study
SNL	speed no load
TDH	total dynamic head
WAC	Washington State Administrative Code
WBS	Work Breakdown Structure
WES	Waterways Experiment Station
WSDOT	Washington State Department of Transportation

Units of Measure

cfs	cubic feet per second
cy	cubic yard
ft	feet
ft/s	feet per second
gpm	gallons per minute
m	meter
m/s	meter per second
m^3	cubic meter
m^3/s	cubic meters per second
mm	millimeter

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Executive Summary

The purpose of the Lower Snake River Juvenile Salmon Migration Feasibility Study (Feasibility Study) is to evaluate modifications to existing dam and reservoir facilities that may increase the survival of juvenile anadromous fish as they migrate through the four hydropower facilities of the U.S. Army Corps of Engineers (Corps) Lower Snake River Project. The three pathways presented in the Feasibility Study are: 1) a combination of major system improvements, 2) a surface bypass and collection program, and 3) a permanent drawdown of the four lower Snake River reservoirs. The Feasibility Study contains numerous appendices that specifically address the economic, biological, social, and engineering aspects of each of these pathways. This appendix, the Natural River Drawdown Engineering Appendix, summarizes the process necessary to implement the drawdown pathway.

The implementation of drawdown of the four Lower Snake River reservoirs requires that five steps be completed: 1) The water in each reservoir must be evacuated, 2) a portion of each dam must be removed to allow the entire river to flow freely 3) a means to channelize the river is necessary to control the river as it flows around the abandoned structures, 4) the abandoned structures must be decommissioned and each facility secured against public access, and 5) numerous structures and facilities in the reservoirs must be modified in order to operate with lower water surface elevations. Two primary criteria guided the development, evaluation, and selection of the engineering alternatives presented in this report:

- Selected measures must benefit the survival of the species.
- The least costly, functionally appropriate alternative should be selected.
- Logical and reasonable modifications and construction operations should be selected.
- Operations must be structured to provide safe working conditions and safe river conditions.

To draw down the reservoirs, the Corps must modify turbines and turbine passages to allow them to be used as low-level outlets. Even though the outlets would operate for only 60 to 90 days while the embankment is excavated to create a new channel, the facility must function properly during this period or risk catastrophic failure of the embankments and other structures.

Embankment removal would be performed concurrently with reservoir drawdown using commercially available, large-capacity excavation and hauling equipment. Over 9 million cubic meters (m^3) (12 million cubic yards [cy]) of material must be excavated and removed to stockpile areas. The work would be performed during the time period between the end of the spill season in August and the start of the next high flow season in January. The construction of channelization levees would immediately follow and be completed in March of the same season. To construct pervious levees, over 1.8 million m^3 (2.4 million cy) of shotrock and riprap must be hauled by barge to the four project sites.

The concrete structures such as the powerhouses, navigation locks, and the non-overflow dams would remain within the channelization levees and would be secured against public access. Disposition of the remaining steel structures would include excessing, salvaging, and abandoning these structures. Similar treatment for mechanical and electrical equipment was investigated. Since most of the equipment is nearing the end of its service life, no cost benefits for salvaged equipment

these structures. Similar treatment for mechanical and electrical equipment was investigated. Since most of the equipment is nearing the end of its service life, no cost benefits for salvaged equipment were assumed. The study also investigated methods and costs of demolishing the remaining concrete structures but did not recommend that operation because it was too costly.

Modification of the reservoir infrastructure would be necessary as a result of lowering the reservoirs. These include the following:

- Up to 25 bridge piers must be protected from erosion due to higher velocity river water.
- Railroad and highway embankments must be protected from erosion due to higher velocity river flows and flows through drainage structures down the exposed surfaces.
- After drawdown is completed, repairs to roads and rail beds would be needed as a result of settlement and slope failures of embankments.

Potential modifications in each reservoir related to fish, wildlife, recreation, and cultural resources include the following:

- Extensive modifications to the Lyons Ferry Hatchery to maintain production during drawdown
- Alternate irrigation facilities at habitat management units to maintain a short-term operation
- Measures to revegetate the exposed land mass and re-establish boundary fencing to promote habitat development
- Modification and, in some cases, closure of recreation areas as a result of drawdown
- A significant cultural resources protection program is planned to protect over 375 known sites that will be exposed after drawdown.

A number of major agricultural and industrial modifications would be needed by drawdown. These measures are not included in the implementation plan, but are part of the economic evaluation. They include:

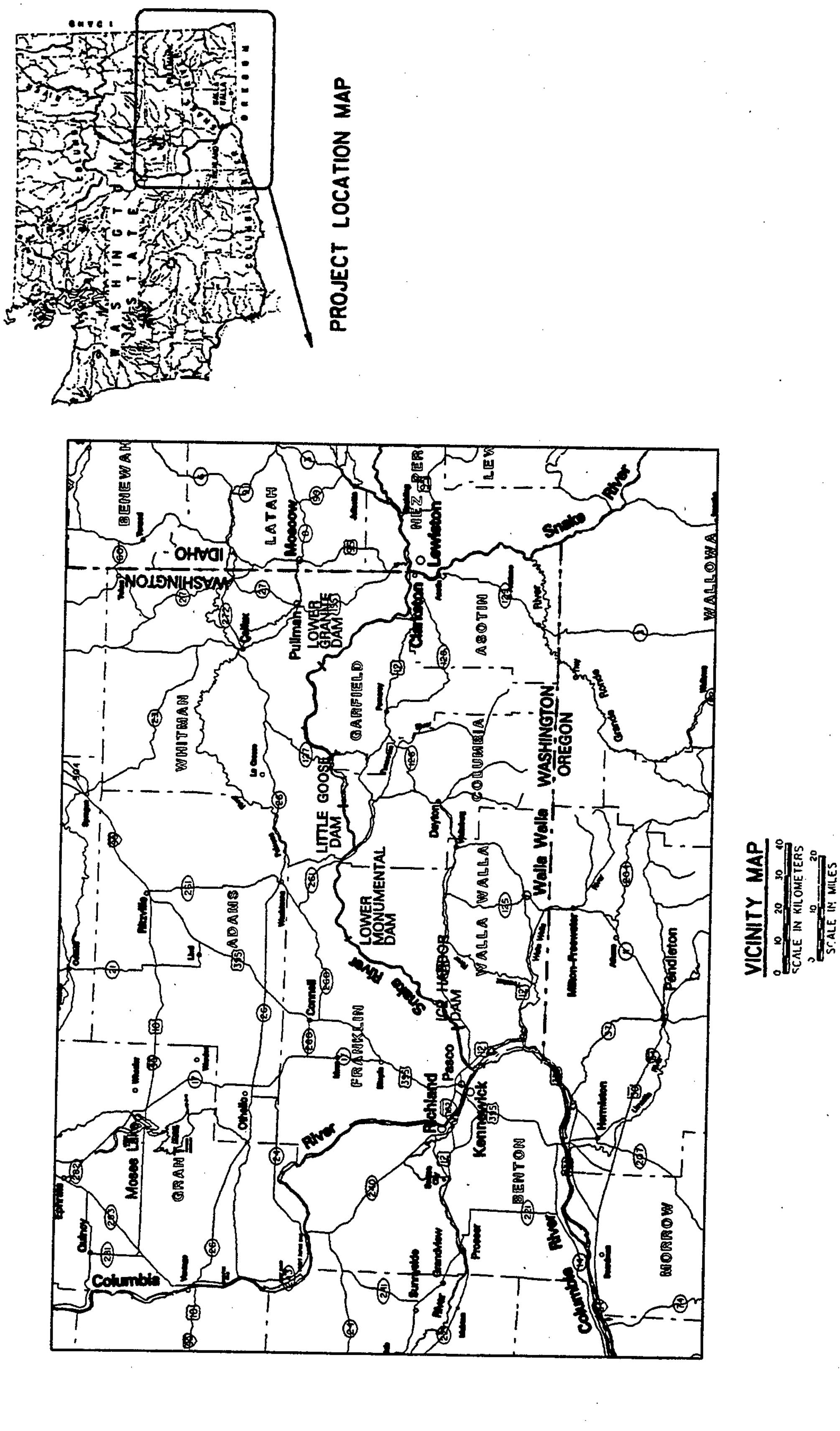
- Concepts for a corporate irrigation system for the major irrigators now using the Ice Harbor Reservoir
- Water intakes for industrial and municipal use
- An industrial effluent diffuser
- A modified river crossing for a gas pipeline
- Modifications to existing water wells.

The recommended sequence for implementing drawdown is to concurrently breach Lower Granite and Little Goose dams in one construction season followed by concurrent breach of Lower Monumental and Ice Harbor dams the following construction season. Numerous engineering and construction activities must precede the dam breaching as well as follow dam breaching. The timeframe for implementing drawdown of the four lower Snake River dams is estimated to extend over 9 years with full funding.

1. Introduction

This appendix describes the process necessary for implementing a permanent drawdown of four dams on the lower Snake River: Lower Granite, Little Goose, Lower Monumental, and Ice Harbor. Figure 1-1 provides a regional map showing the location of each of the facilities. The Corps' study team considered a number of options in selecting the methods and procedures necessary to implement the drawdown and mitigate its effects on infrastructure; natural, recreational, and cultural resources; and agricultural and industrial operations. This appendix summarizes those options and the rationale for them and provides a comprehensive plan for implementing a permanent drawdown. The major elements of drawdown are: 1) Reservoir Evacuation, 2) Embankment Removal, 3) River Channelization, 4) Reservoir Modifications, and 5) Hydropower Facilities Decommissioning. These elements are shown in Figure 1-2.

In addition, work continues on the two other designated pathways to improve salmon survival related to the Lower Snake River Hydropower Project: 1) a surface bypass and collection program and 2) other major system improvements. Those pathways are documented in other appendices in this Feasibility Study.



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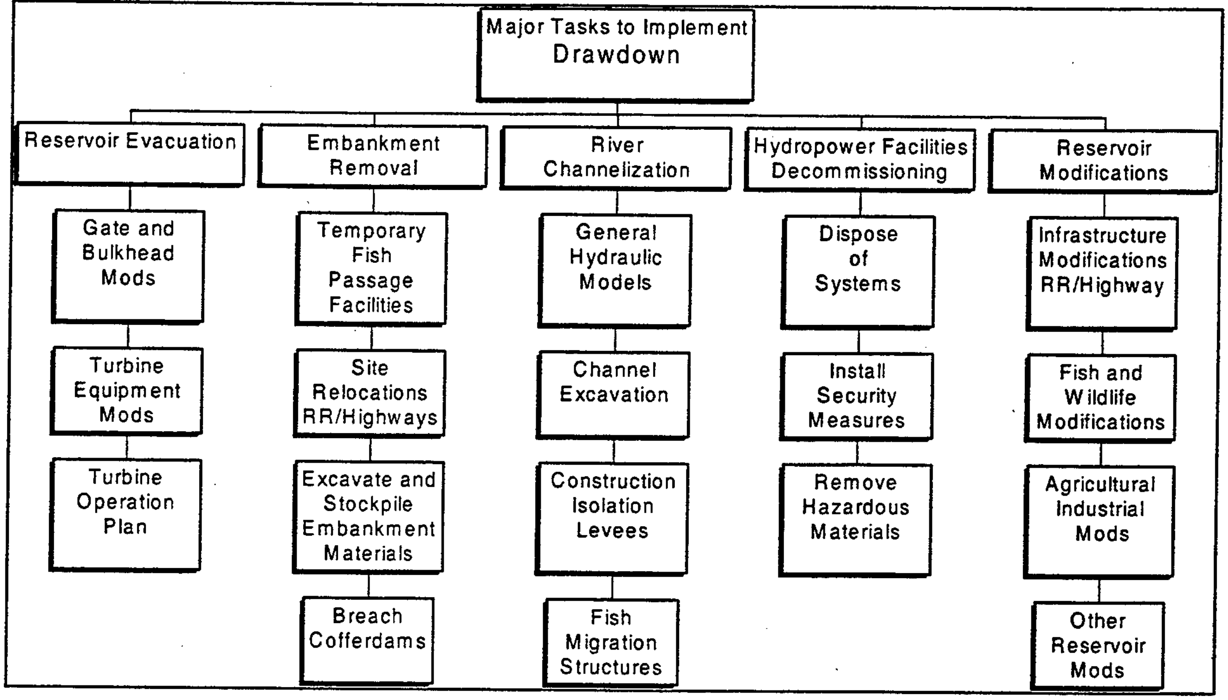
Walla Walla District

Figure 1-1. Location and Vicinity Maps of Lower Snake River Facilities

Figure 1-2. Elements of the Comprehensive Plan for Implementing Permanent Drawdown



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2. Background

2.1 General

On March 2, 1995, the National Marine Fisheries Service (NMFS) issued a Biological Opinion for the *Reinitiation of Consultation on 1994-1998 Operation of the Federal Columbia River Power System and Juvenile Transportation Program in 1995 and Future Years* (NMFS, 1995). The Biological Opinion established immediate measures necessary for the survival and recovery of Snake River salmon stocks listed under the Endangered Species Act (ESA). A specific decision path for the implementation of long-term alternatives was also identified.

This path identified two major decision points. The first decision point was in 1996 and required an interim status report with a preliminary decision regarding the selection of one of three drawdown alternatives for the lower Snake River in order to proceed with detailed engineering or the elimination of any further consideration of drawdown. In case a decision on drawdown could not be reached in 1996, a second decision point was identified in 1999. At that time, a final plan for drawdown or surface bypass and collection was to be selected, and feasibility evaluations and National Environmental Policy Act (NEPA) documentation were to be completed.

2.2 The Evolution of this Study

The Corps' response to the Biological Opinion issued by the NMFS and, ultimately, this Feasibility Study evolved from a System Configuration Study (SCS) initiated in 1991.

The SCS was undertaken to evaluate the technical, environmental, and economic effects of potential modifications to the configuration of Federal dams and reservoirs on the Snake and Columbia rivers to improve survival rates for anadromous salmonids. This process began in response to the Northwest Power Planning Council's *Fish and Wildlife Program Amendments (Phase Two)*, issued in December 1991 (NPPC, 1991). The Phase I SCS, *Columbia River Salmon Migration Analysis, System Configuration Study, Phase I*, assessed various possible alternatives for improving conditions for anadromous salmonid migration and was to be conducted in two separate phases (Corps, 1994).

Phase I of the SCS was completed in June 1995. This was a reconnaissance-level assessment of multiple concepts, including drawdown, upstream collection, additional reservoir storage, a migratory canal, and several other alternatives. Alternatives that displayed the most potential benefit to anadromous fish were carried into Phase II.

Since 1995, Phase II has developed into a major program containing many separate and specific studies. Structural changes for juvenile salmon migration improvements within the lower Snake River are only a portion of the total program. This growth in the scope of Phase II was considered necessary to adequately and efficiently respond to the requirements for multiple evaluations addressed in the Biological Opinion.

In December 1996, the Corps issued the *System Configuration Study, Phase II, Lower Snake River Juvenile Salmon Migration Feasibility Study, Interim Status Report* (Corps, 1996a) in response to the Biological Opinion requirement for a preliminary decision regarding the selection of drawdown alternatives. Between the genesis of the SCS in 1991 and the interim status report in 1996, the

Corps narrowed the drawdown alternatives from 22 that were initially formulated to one called the Natural River Option. The interim status report recommended the Natural River Option as the only drawdown option for further development, basing this recommendation on estimated biological effectiveness, other environmental effects, technical feasibility, cost, and regional acceptance.

The interim status report estimated that the Natural River Option would have the lowest construction cost ($533 million) and the shortest implementation time (5 years) of the primary options under consideration. The report also pointed out, however, that permanent natural river drawdown completely eliminates power production on the lower Snake River, as well as commercial navigation between Lewiston, Idaho, and Pasco, Washington. Cultural resource damage due to the uncovering of sites would be detrimental initially. However, erosion caused by annual reservoir fluctuations would not occur, and sites would eventually be protected by revegetation. Although other environmental impacts are initially substantial, maintaining natural river elevations would allow the ecosystem to achieve equilibrium in the future.

2.3 Drawdown Engineering Study Scope

The concepts, processes, and cost estimates described in this appendix support the single drawdown option considered in this second phase of the Feasibility Study – the Natural River Drawdown Option that was recommended in the interim status report. As mentioned earlier, other options were considered in the Phase I SCS report. The development of concept designs for this Natural River Option engineering study and implementation plan were intended to be done at a feasibility level of design, resulting in a baseline cost estimate. Figures 2-1 through 2-8 provide a plan view drawing and aerial photograph of each of the Lower Snake River projects. Figure 2-9a through 2-9h is a full-system map locating many of the reservoir facilities discussed in this Appendix. Figure 2-10 provides a Drawdown Implementation Schedule Summary. Figure 2-11 provides an estimate project cost for Option A-3a, drawdown dam removal, channel bypass.

2.4 Study Team Composition

The study team consisted of a multidiscipline group from various organizations. The initial team was comprised of Corps of Engineers personnel from various engineering division workgroups. This group formulated the Natural River Option documented in the interim status report. Raytheon Engineers and Constructors, teamed with American Hydro, was added to the team to provide a concept evaluation of the use of turbines and turbine passages for reservoir discharge. Further model evaluations and recommendations were provided by the Corps Waterways Experiment station, Voest-Alpine, and the Corps of Engineers, Hydroelectric Design Center. Raytheon later provided the initial concept design for the embankment excavation, river channelization, and the reservoir infrastructure modifications. Thomas, Dean and Hoskins developed the concept design for the modifications for Potlatch Corporation Water intake and effluent diffuser and the PGE Gasline River Crossing. The remaining modifications were developed by engineers and scientists of the Walla Walla District, Corps of Engineers. A team of engineers from Montgomery-Watson Engineers, CH2M Hill Engineers, and Project Time and Cost provided independent technical review of the document.

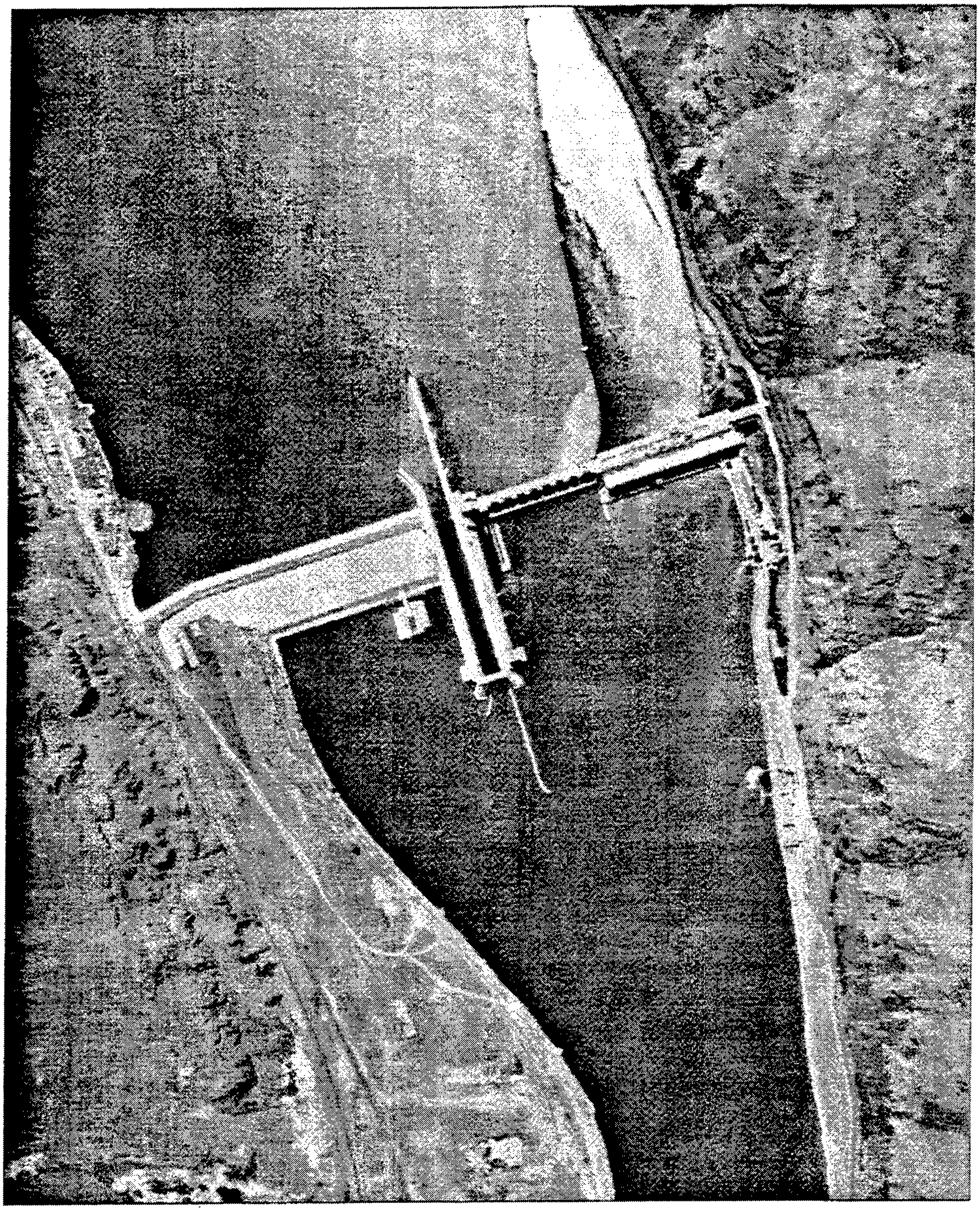

Figure 2-1. Lower Granite Dam – Aerial Photograph

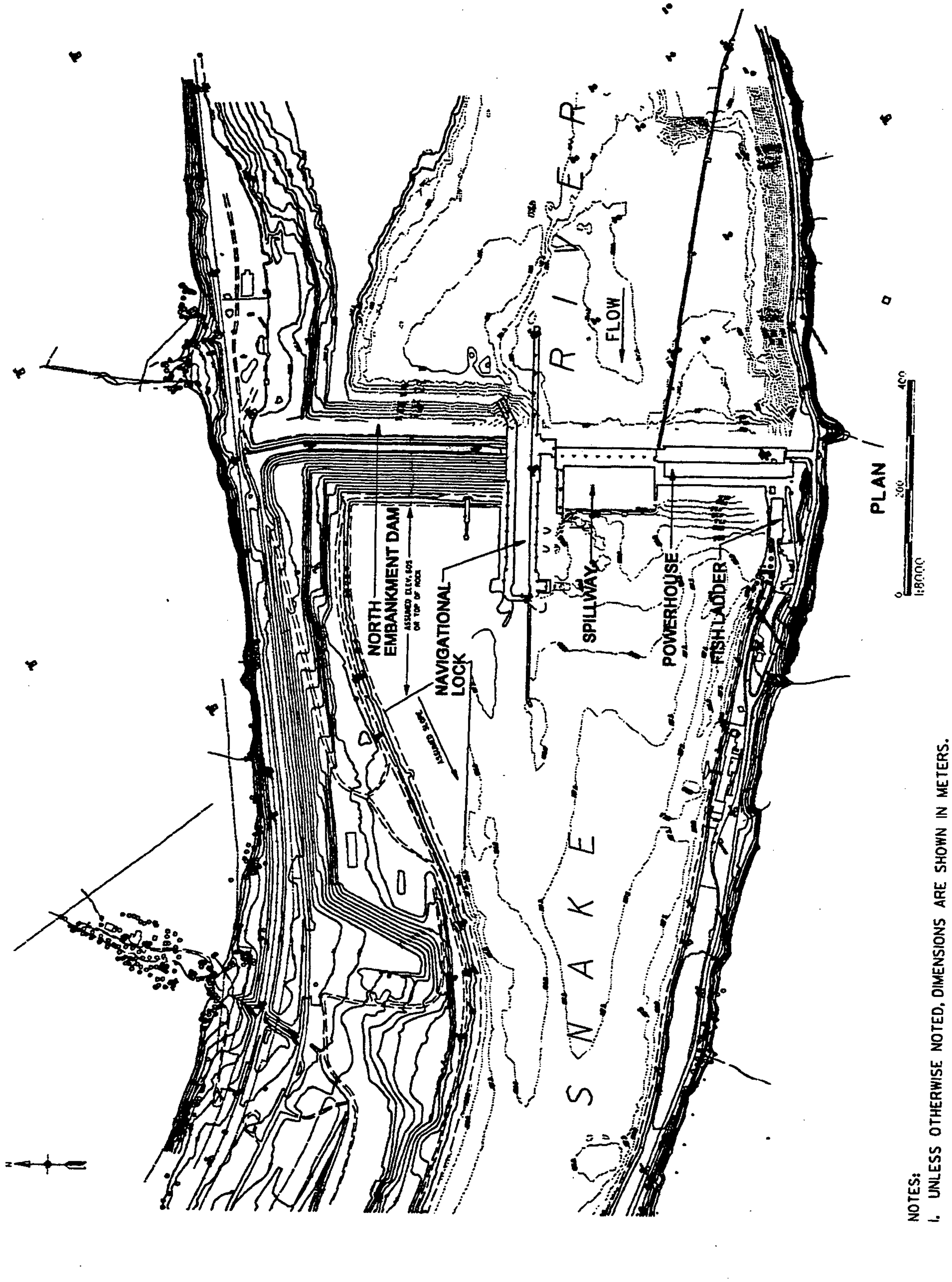

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Figure 2-2. Lower Granite – Existing Project Arrangement General Plan

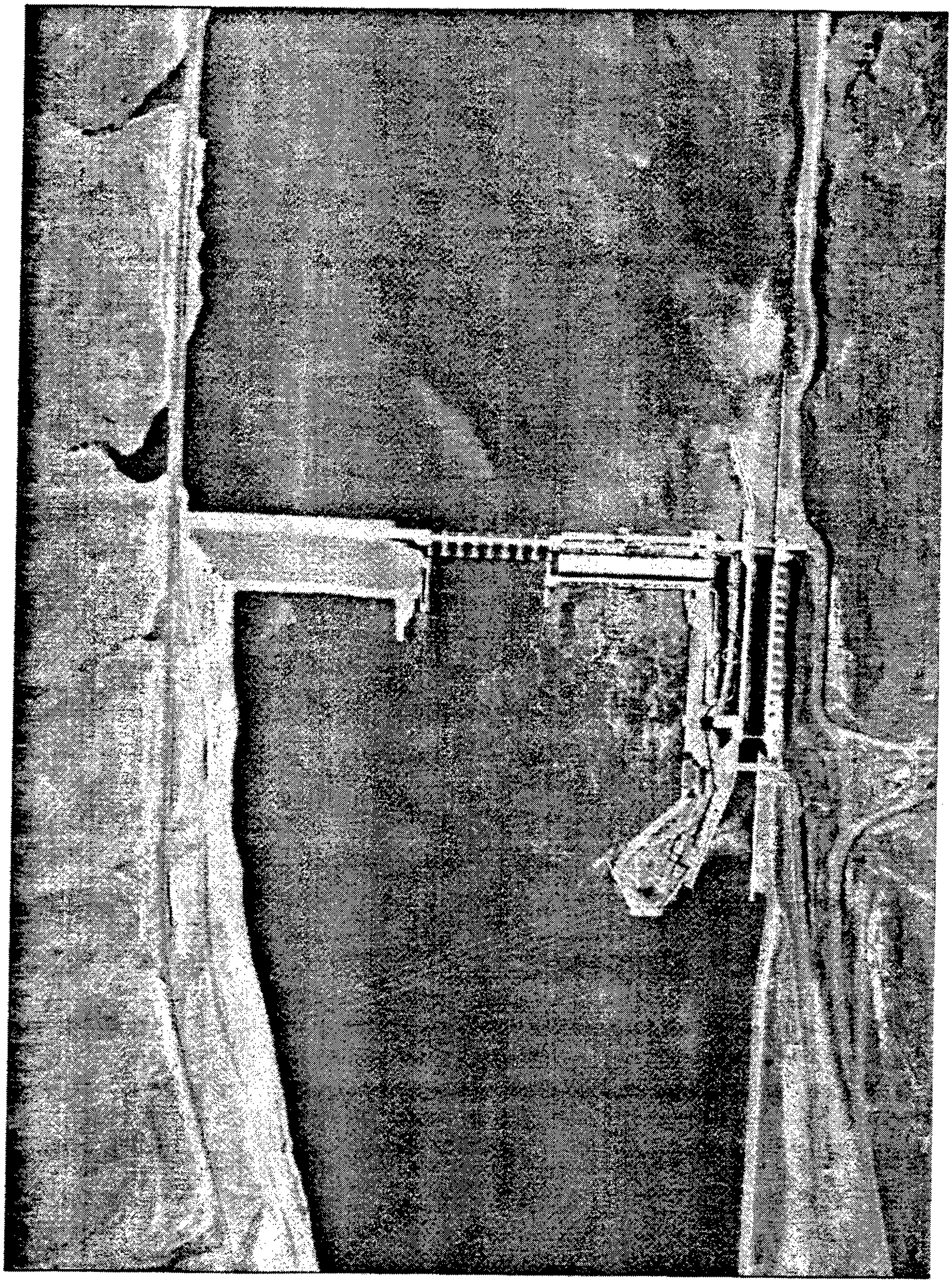

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Figure 2-3. Little Goose Dam – Aerial Photograph

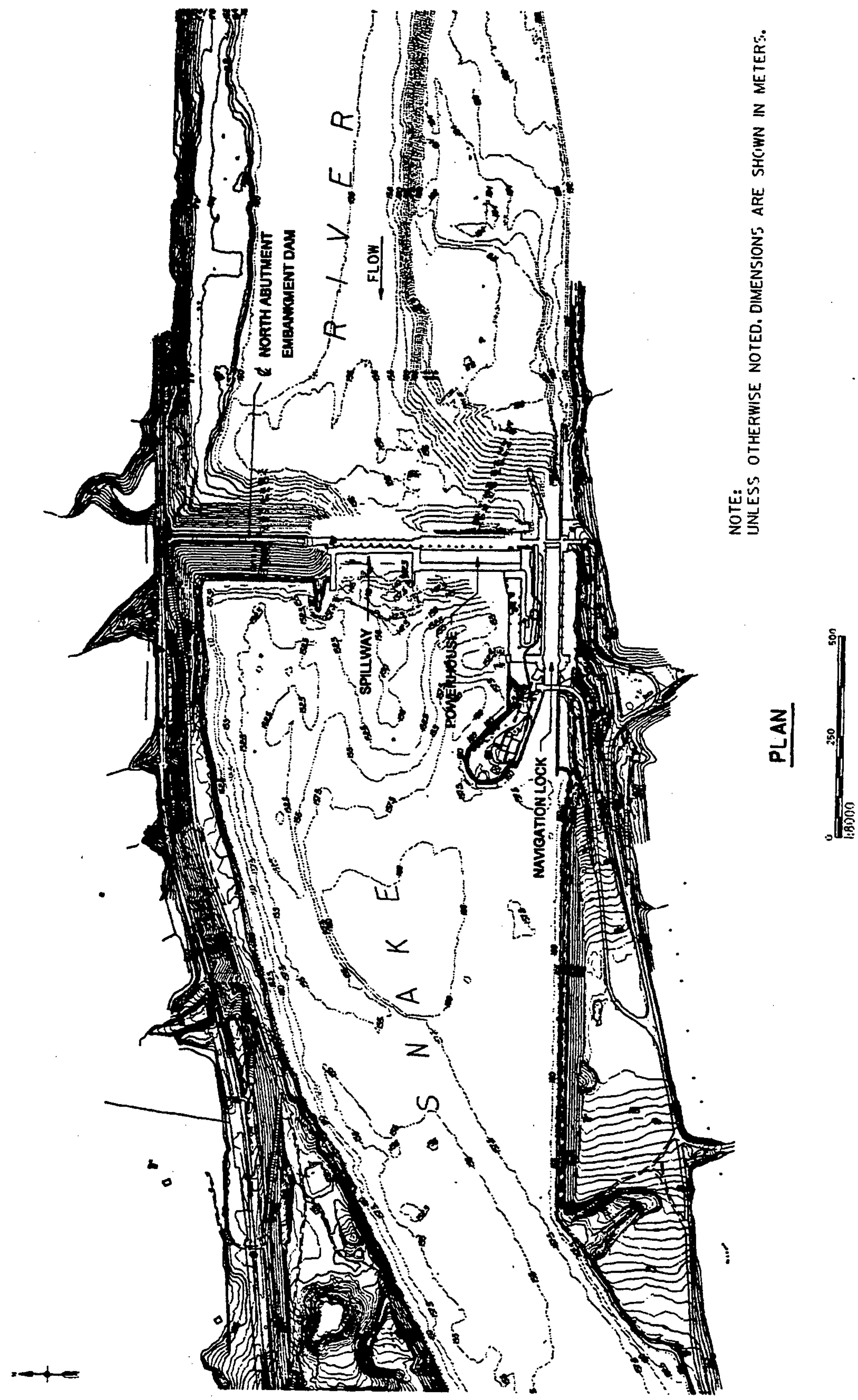

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Figure 2-4. Little Goose – Existing Project Arrangement General Plan

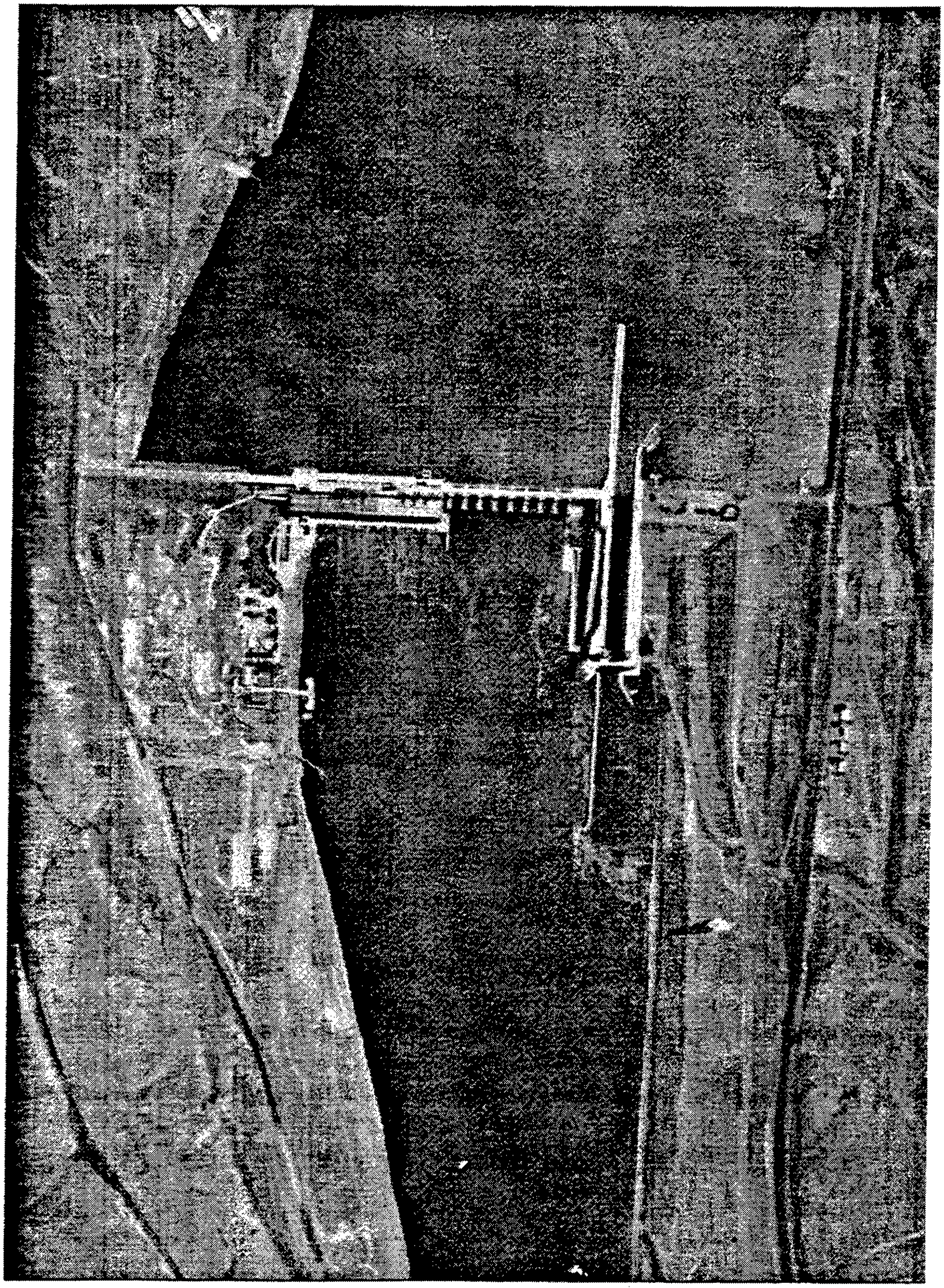

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Figure 2-5. Lower Monumental Dam – Aerial Photograph

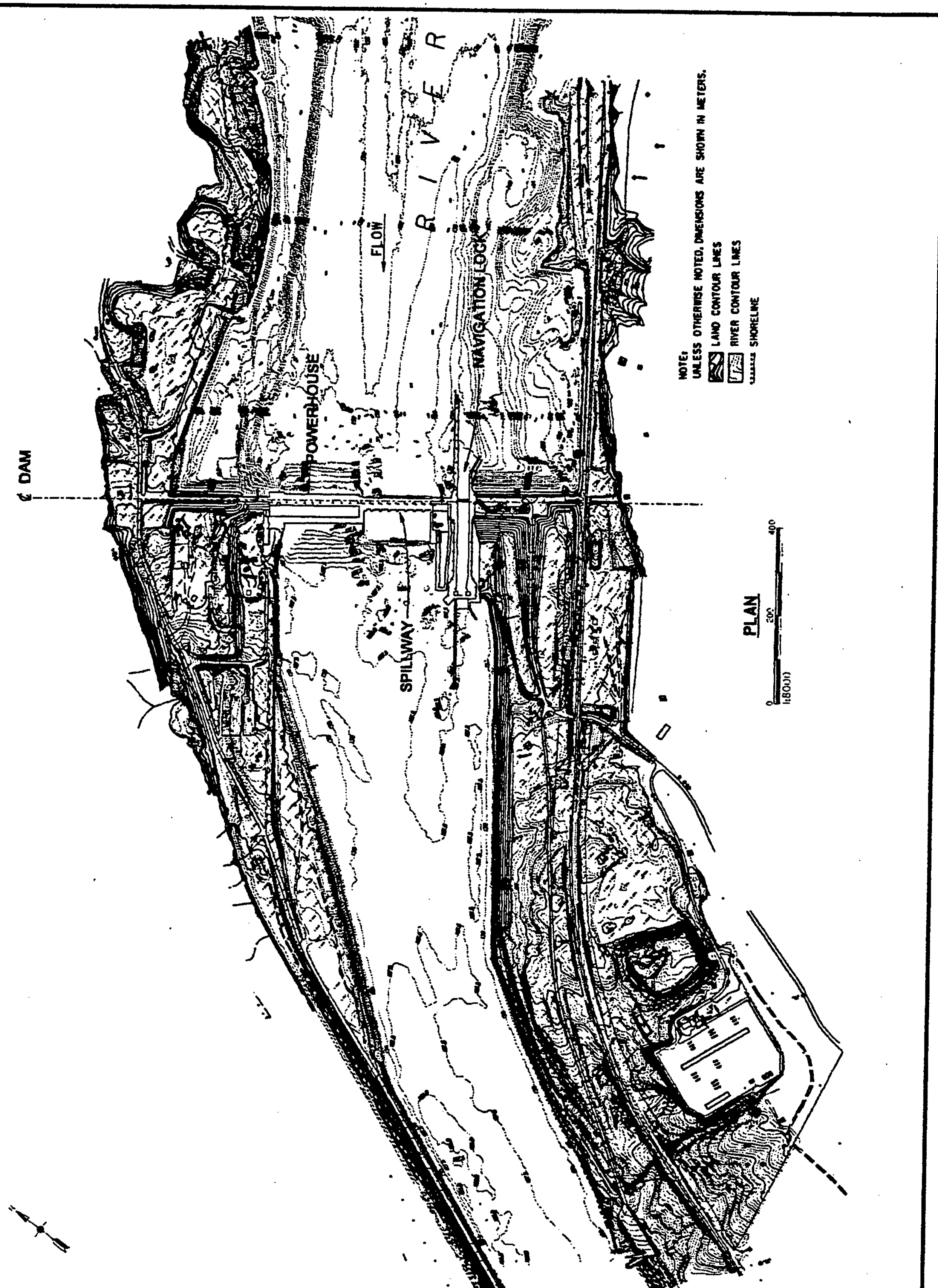

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Figure 2-6. Lower Monumental – Existing Project Arrangement General Plan

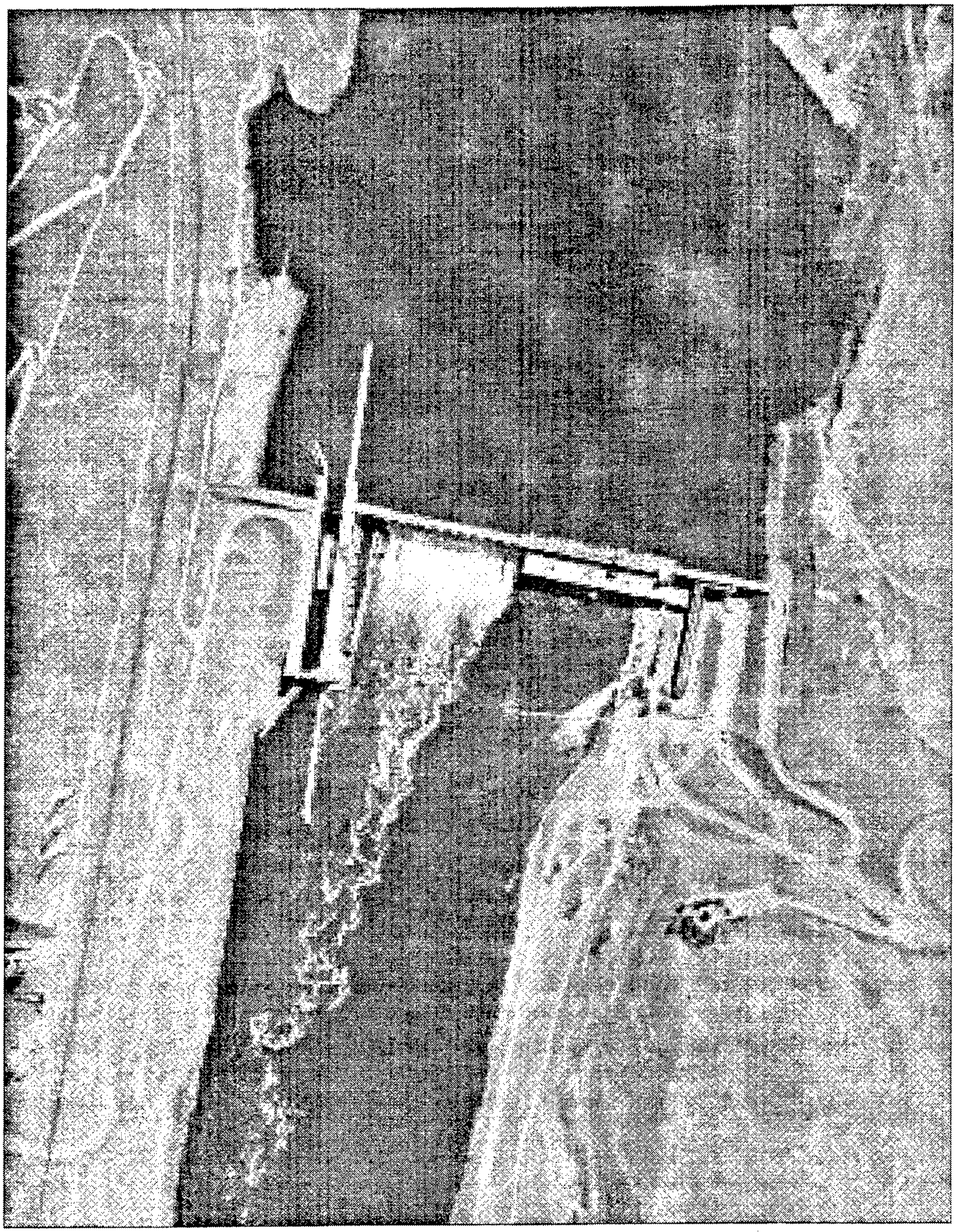

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Figure 2-7. Ice Harbor Dam – Aerial Photograph

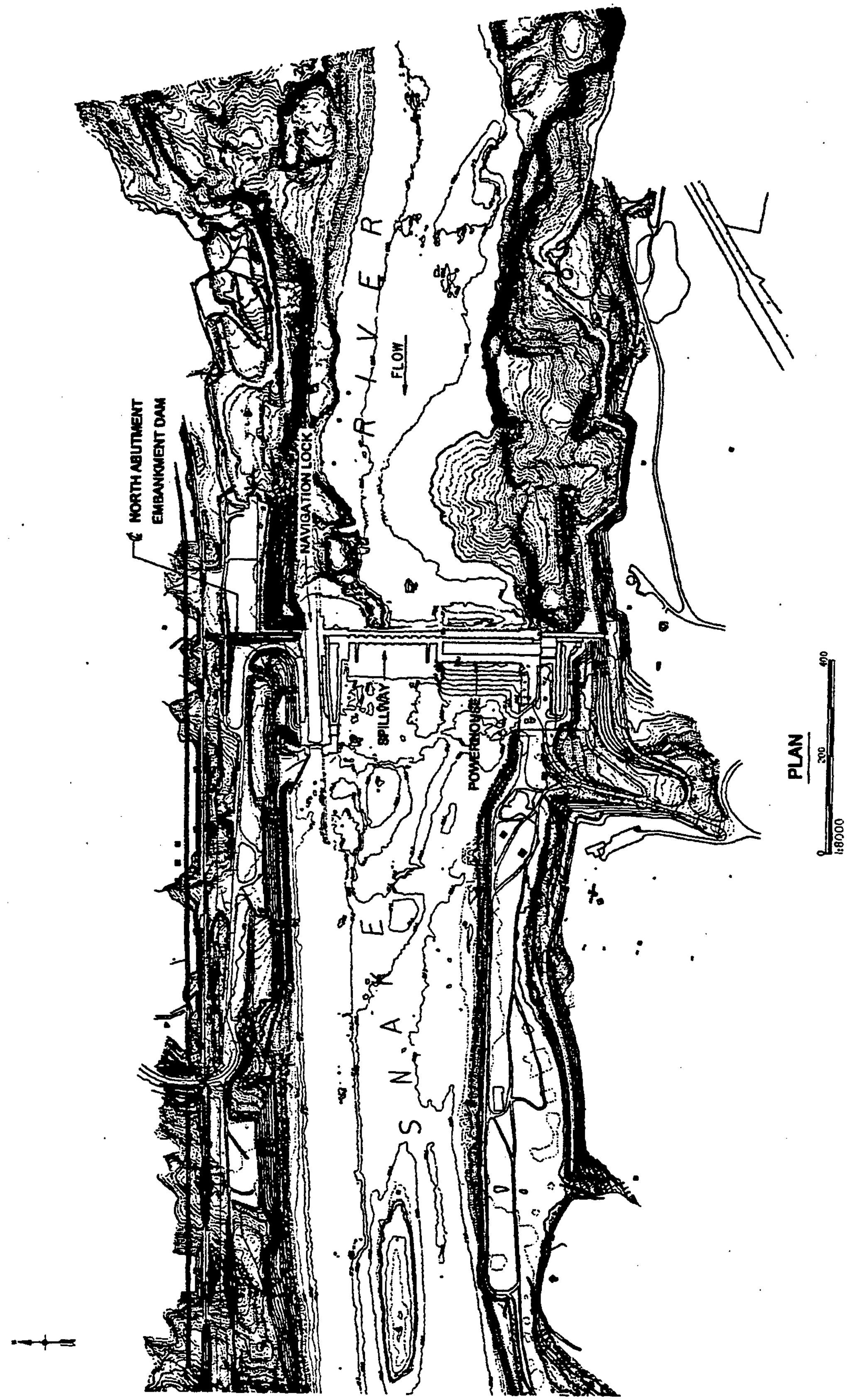

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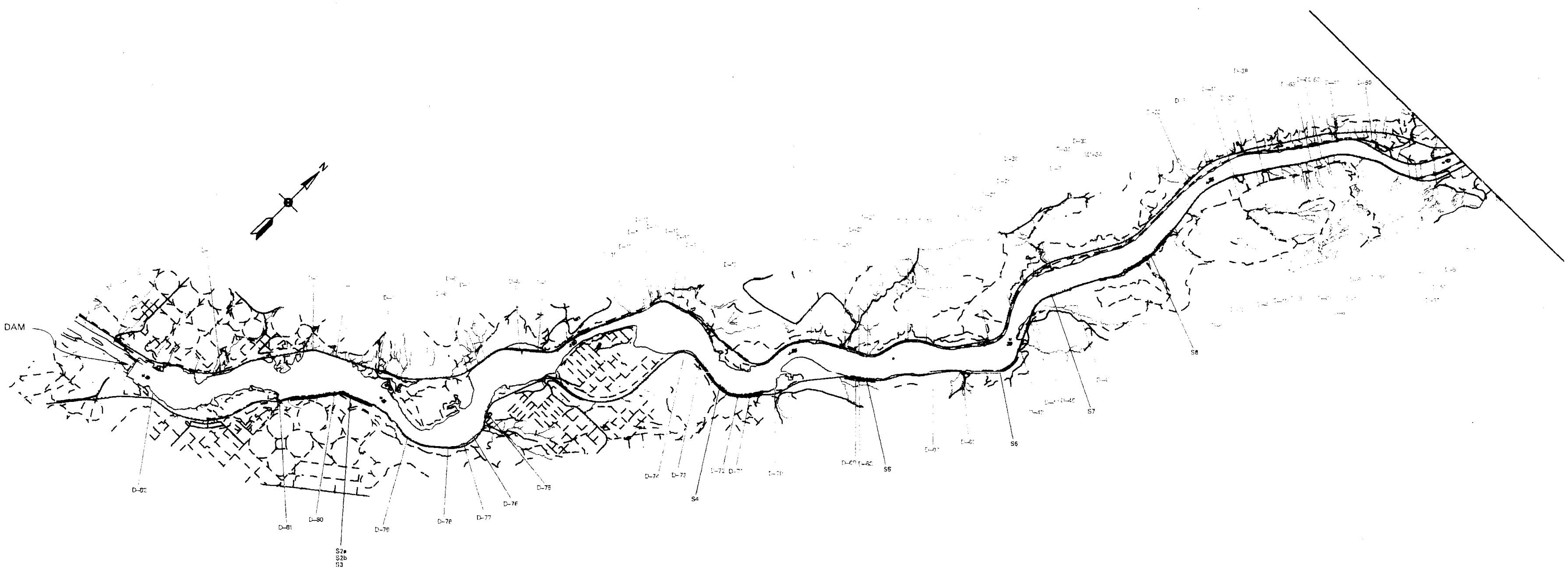
Figure 2-8. Ice Harbor – Existing Project Arrangement General Plan

CORPS OF ENGINEERS SAFETY PAYS U. S. ARMY

LAKE SACAJAWEA (ICE HARBOR POOL)



LEGEND

D-45 DRAINAGE STRUCTURES

EMBANKMENT RIPRAP LOCATIONS (i.e. N1, S2)

Figure 2-9a. Snake River – General System Arrangement: Lake Sacajawea (Drainage Structures)

VALUE ENGINEERING PAYS CONT. NO.

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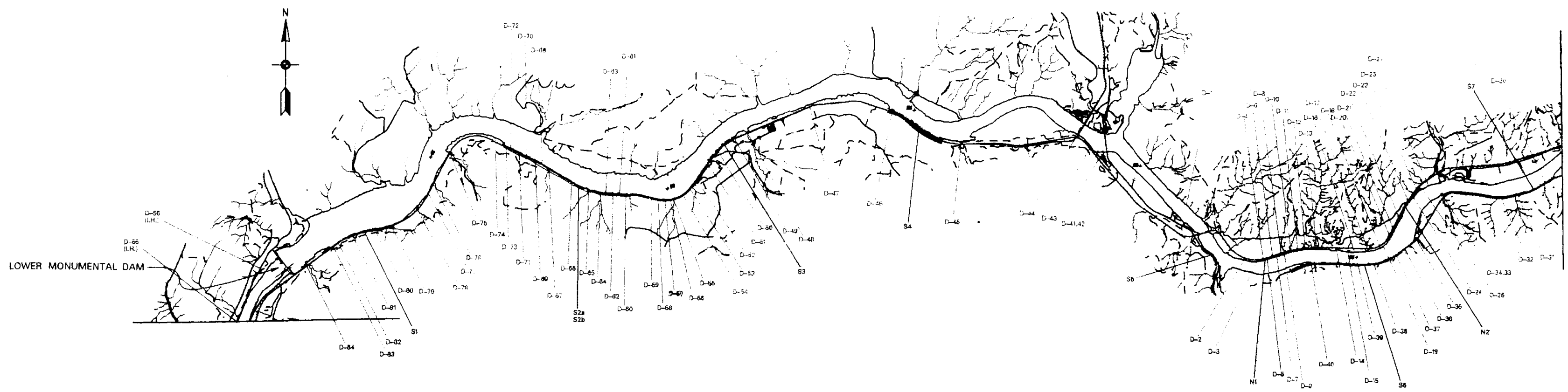
LAKE WEST (LOWER MONUMENTAL POOL)



LEGEND

D-45 DRAINAGE STRUCTURES

EMBANKMENT RIPRAP LOCATIONS (i.e. N1, S2)

Figure 2-9b. Snake River – General System Arrangement: Lake West (Drainage Structures)

VALUE ENGINEERING PAYS

CONT. NO.

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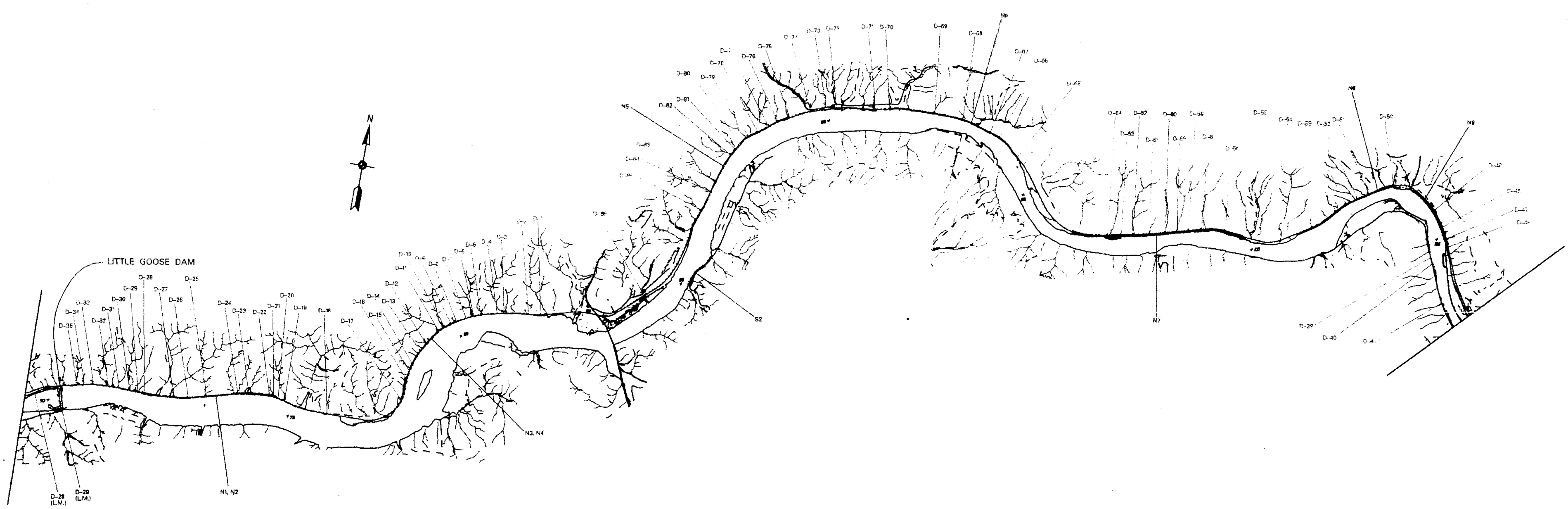
LAKE BRYAN (LITTLE GOOSE POOL)



LEGEND

D–45 **DRAINAGE STRUCTURES**

■ **EMBANKMENT RIPRAP LOCATIONS (i.e. N1, S2)**

Figure 2-9c. Snake River -- General System Arrangement: Lake Bryan (Drainage Structures)

VALUE ENGINEERING PAYS

CONT. NO.

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LOWER GRANITE LAKE (LOWER GRANITE POOL)

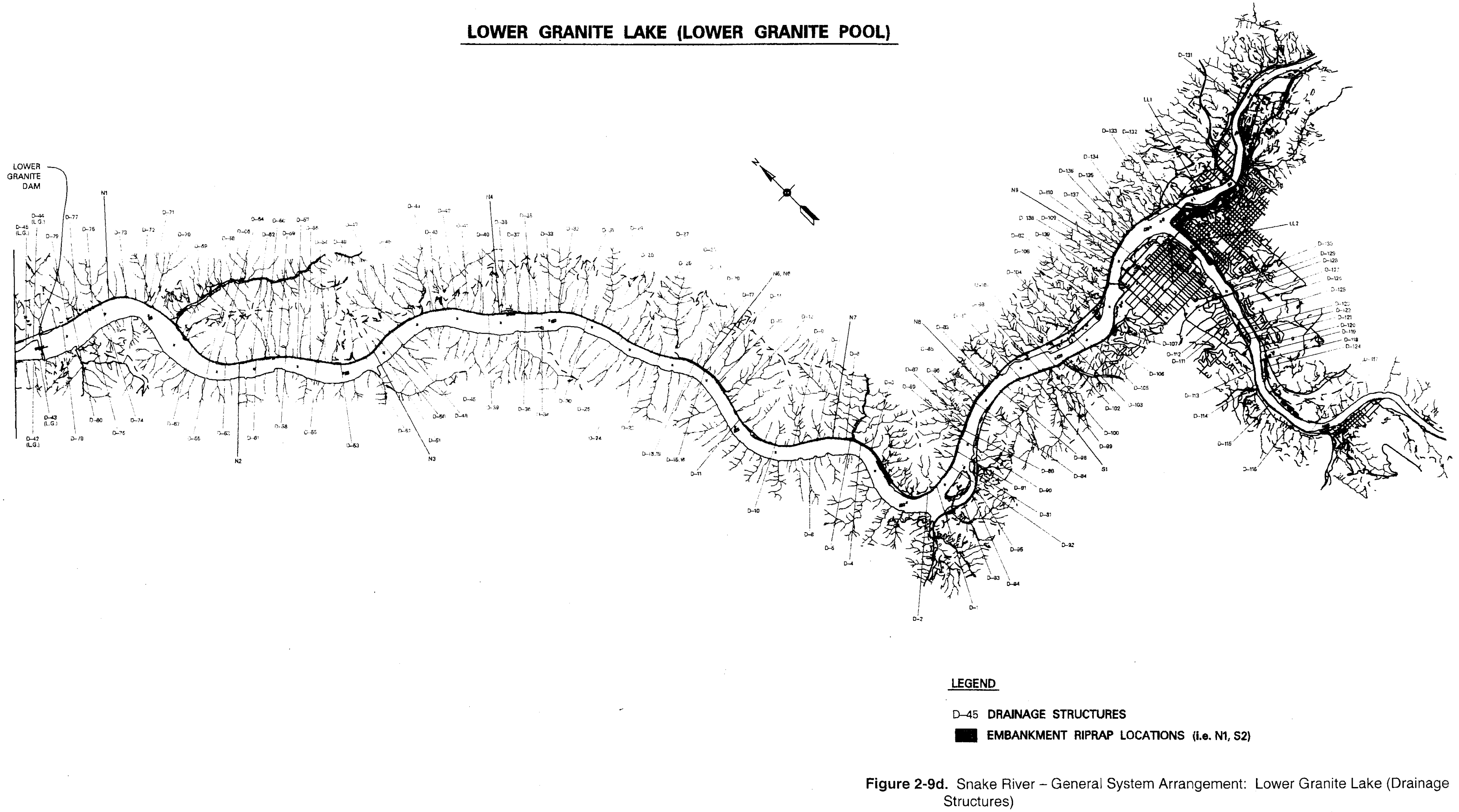

Figure 2-9d. Snake River – General System Arrangement: Lower Granite Lake (Drainage Structures)

VALUE ENGINEERING PAYS

CONT. NO.

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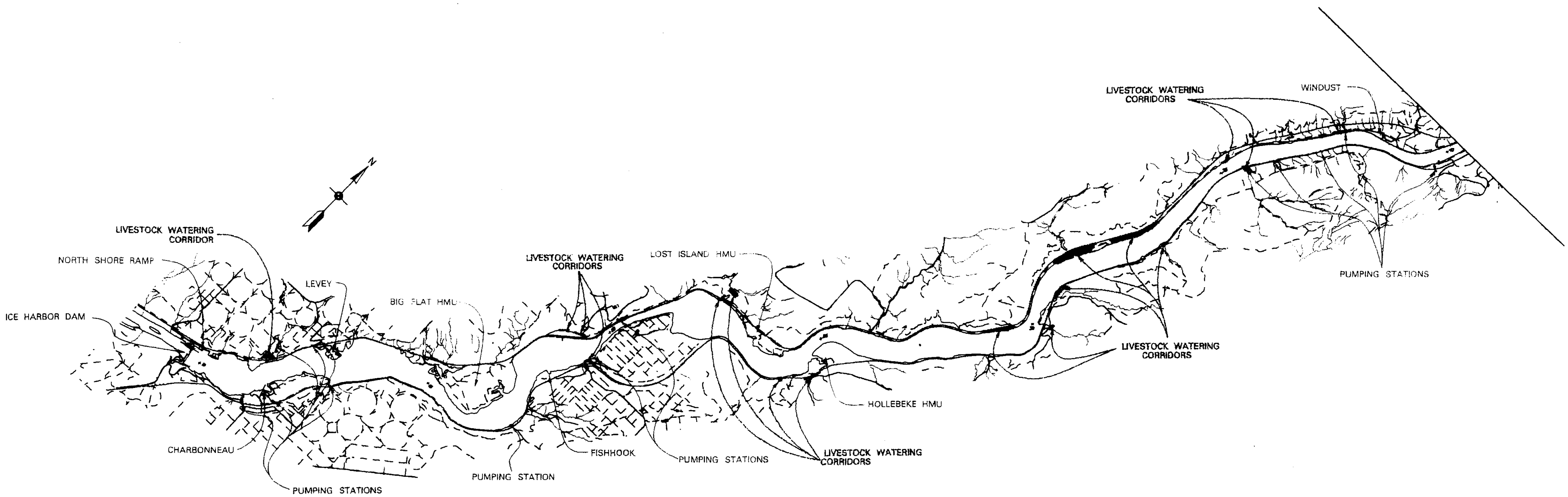
LAKE SACAJAWEA (ICE HARBOR POOL)



LEGEND

DAMS	LYONS FERRY HATCHERY PIPELINE
PARKS & RECR. AREAS	LIVESTOCK WATERING CORRIDORS
HABITAT MGT. UNITS	PUMPING STATIONS & PIPELINES
BRIDGES	CITY WASTE EFFLUENT
PGE GASLINE CROSSING	POTLATCH INTAKE AND DIFFUSER

Figure 2-9e. Snake River – General System Arrangement: Lake Sacajawea (Miscellaneous)

VALUE ENGINEERING PAYS | CONT. NO.

LAKE WEST (LOWER MONUMENTAL POOL)

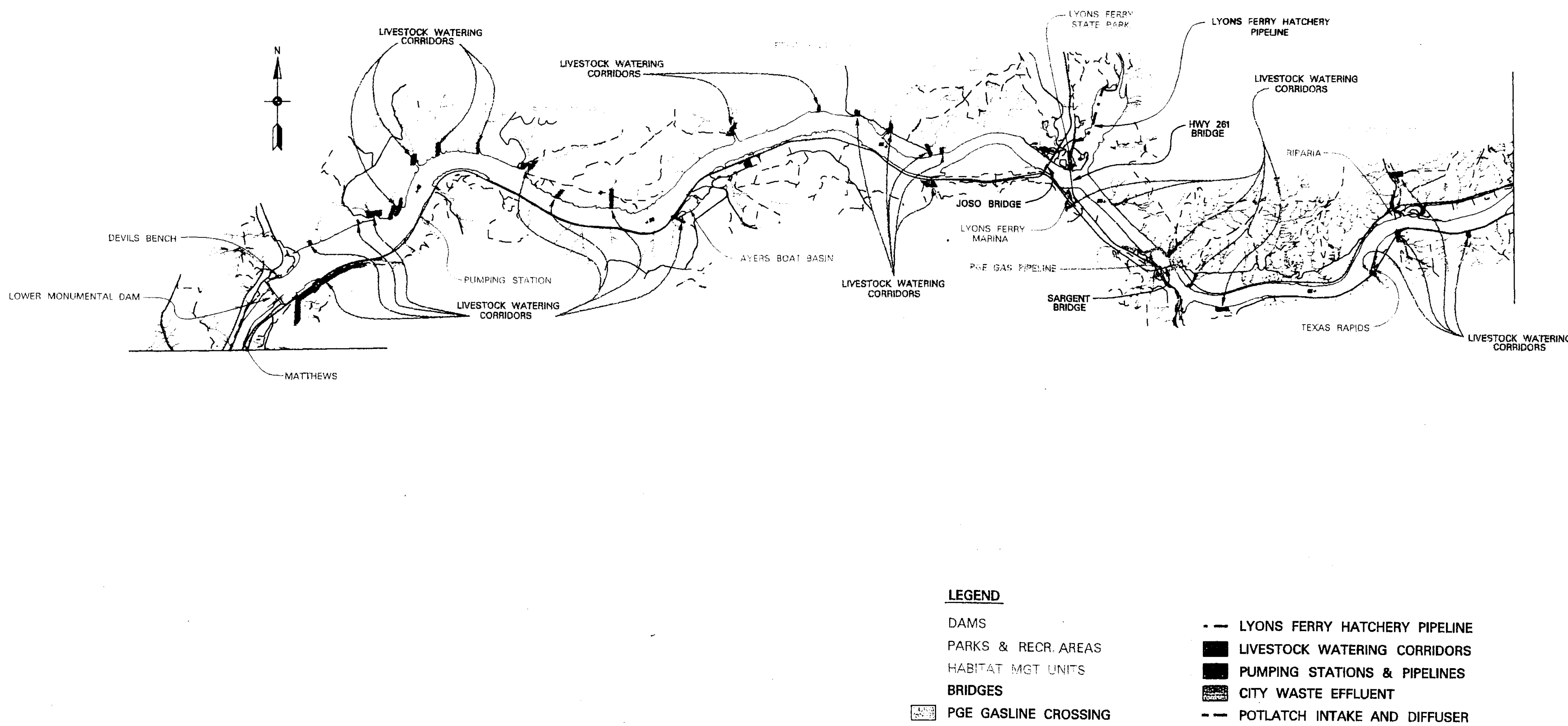


Figure 2-9f. Snake River – General System Arrangement: Lake West (Miscellaneous)

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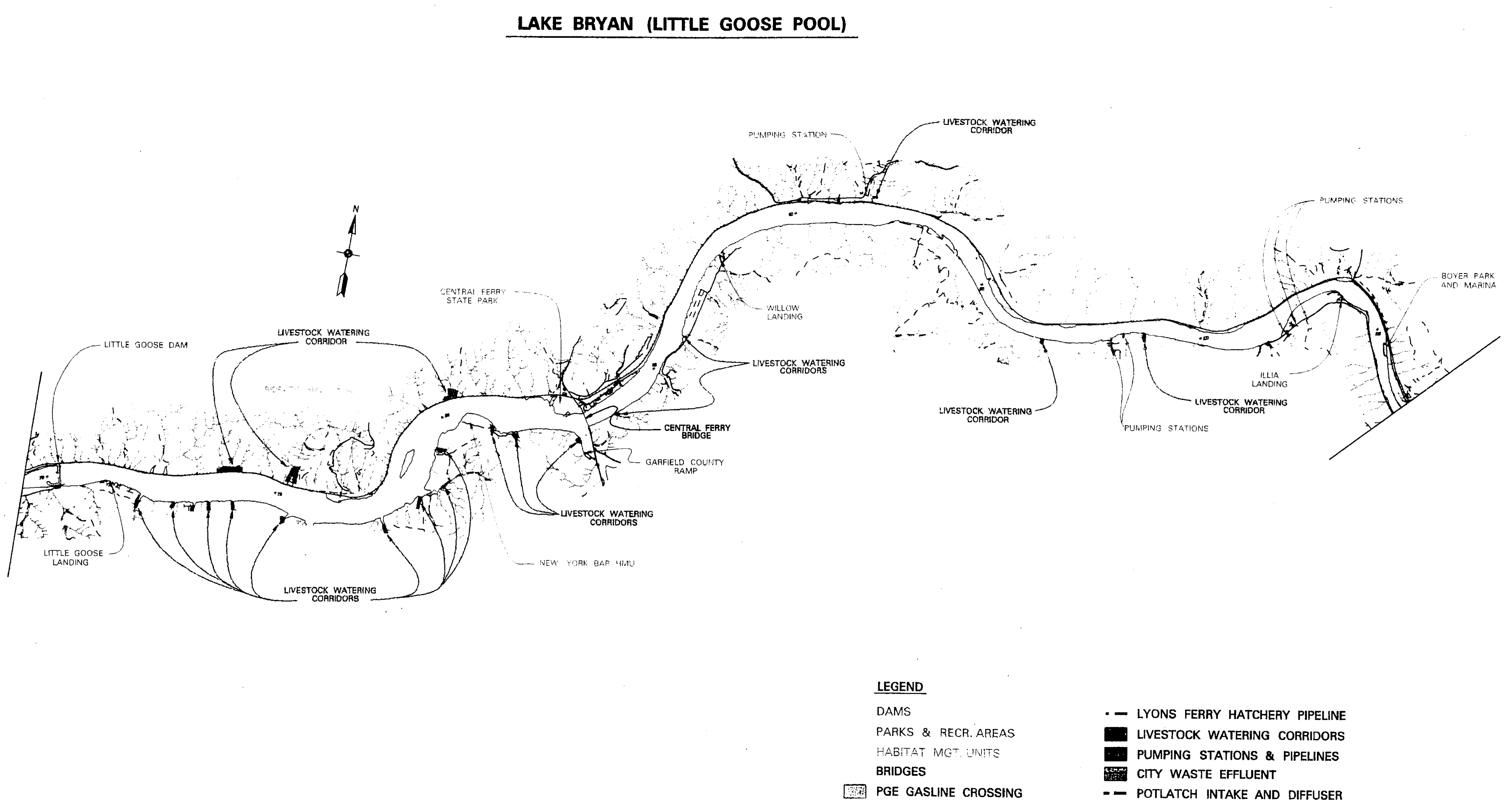

Figure 2-9g. Snake River – General System Arrangement: Lake Bryan (Miscellaneous)

VALUE ENGINEERING PAYS | CONT. NO.

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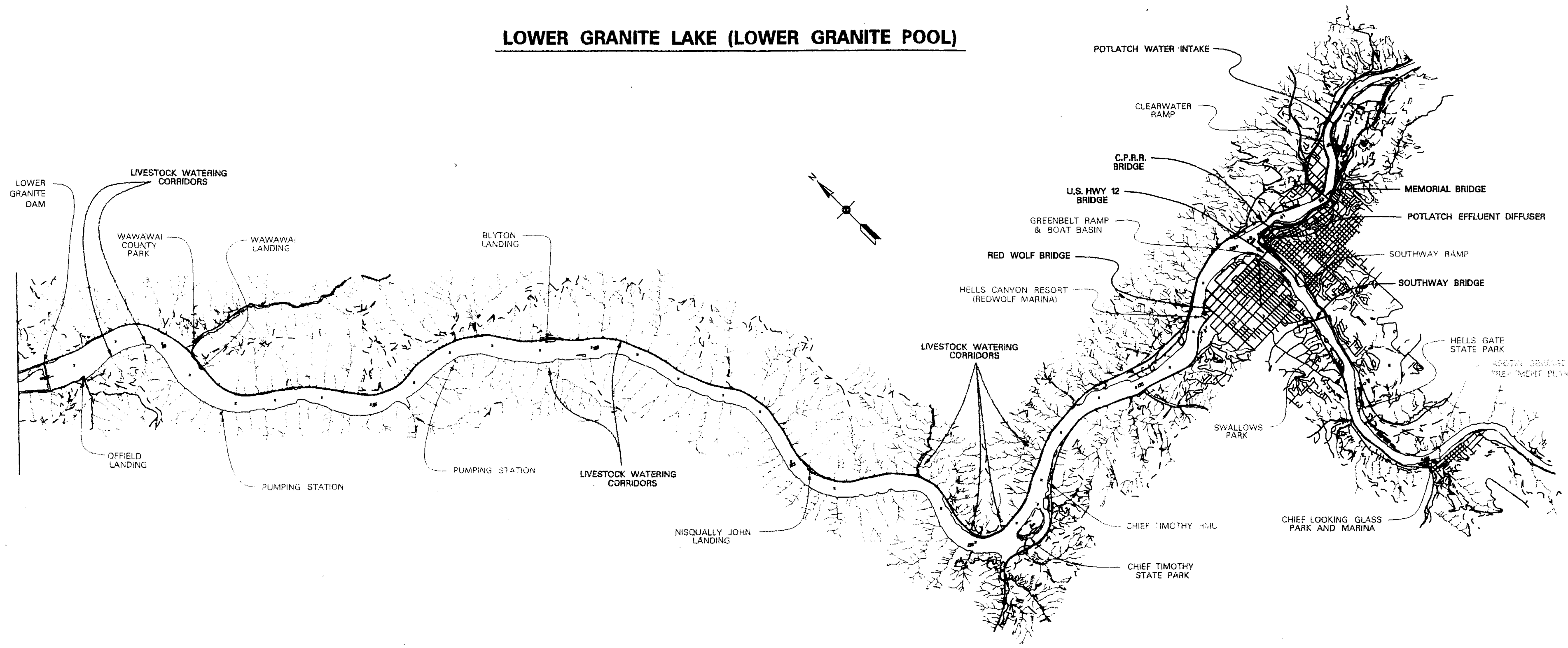

LEGEND

DAMS	LYONS FERRY HATCHERY PIPELINE
PARKS & RECR. AREAS	LIVESTOCK WATERING CORRIDORS
HABITAT MGT. UNITS	PUMPING STATIONS & PIPELINES
BRIDGES	CITY WASTE EFFLUENT
PGE GASLINE CROSSING	POTLATCH INTAKE AND DIFFUSER

Figure 2-9h. Snake River – General System Arrangement: Lower Granite Lake (Miscellaneous)

VALUE ENGINEERING PAYS — CONT. NO.

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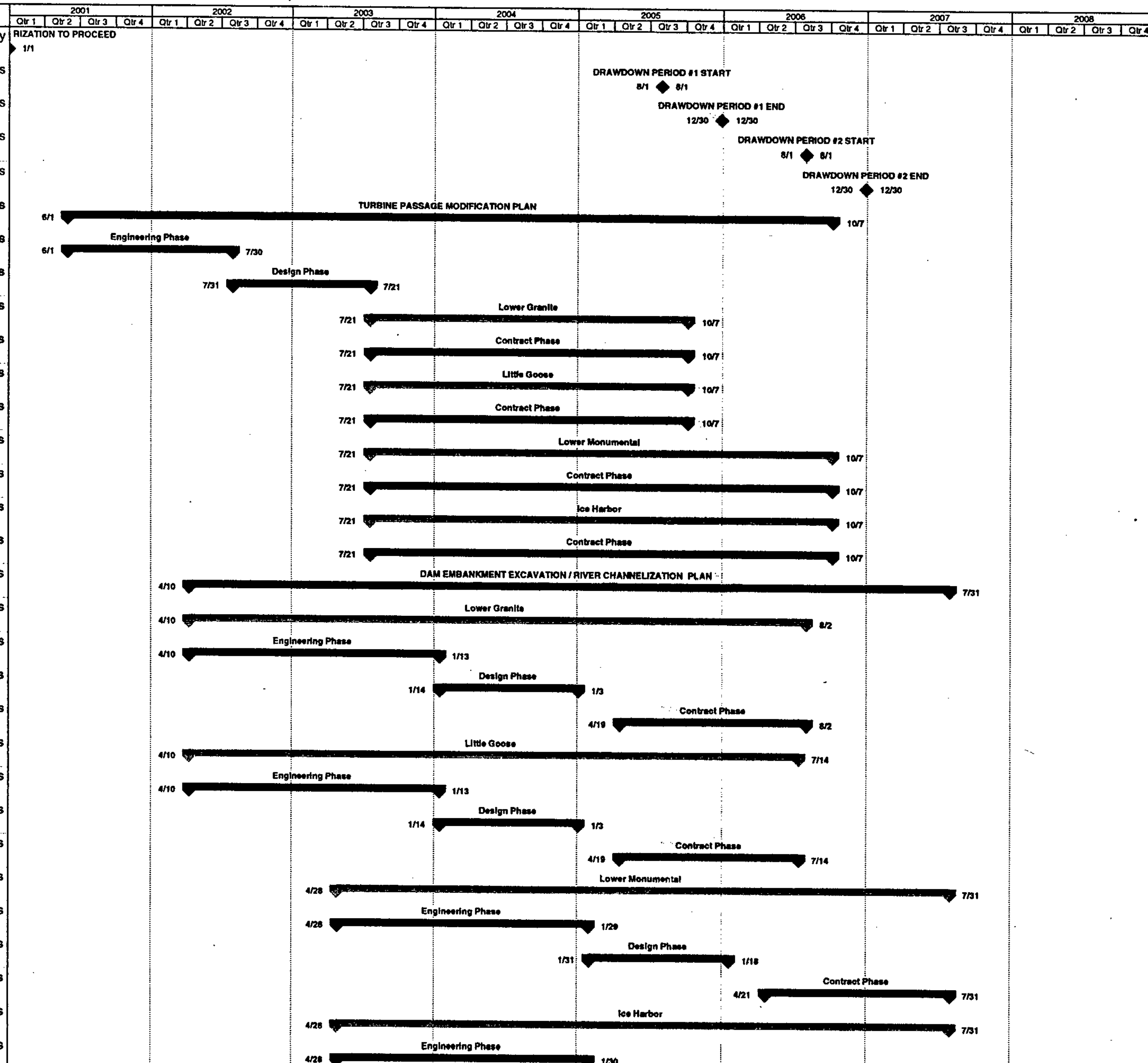
Figure 2-10
Drawdown Implementation Schedule

ID	Task Name	Duration
1	CONGRESSIONAL AUTHORIZATION TO PROCEED	1 day
2	DRAWDOWN PERIOD #1 START	0 days
3	DRAWDOWN PERIOD #1 END	0 days
4	DRAWDOWN PERIOD #2 START	0 days
5	DRAWDOWN PERIOD #2 END	0 days
6	**TURBINE PASSAGE MODIFICATION PLAN**	1694 days
7	Engineering Phase	364 days
11	Design Phase	304.2 days
14	Lower Granite	696.8 days
15	Contract Phase	696.8 days
28	Little Goose	696.8 days
29	Contract Phase	696.8 days
42	Lower Monumental	1025.8 days
43	Contract Phase	1025.8 days
56	Ice Harbor	1025.8 days
57	Contract Phase	1025.8 days
70	**DAM EMBANKMENT EXCAVATION / RIVER CHANNELIZATION PLAN**	1685.5 days
71	Lower Granite	1368 days
72	Engineering Phase	552 days
76	Design Phase	304.5 days
79	Contract Phase	421 days
108	Little Goose	1350.5 days
109	Engineering Phase	552 days
113	Design Phase	304.2 days
116	Contract Phase	403.5 days
145	Lower Monumental	1357.5 days
146	Engineering Phase	552 days
150	Design Phase	305.5 days
153	Contract Phase	409.5 days
183	Ice Harbor	1357.5 days
184	Engineering Phase	552 days

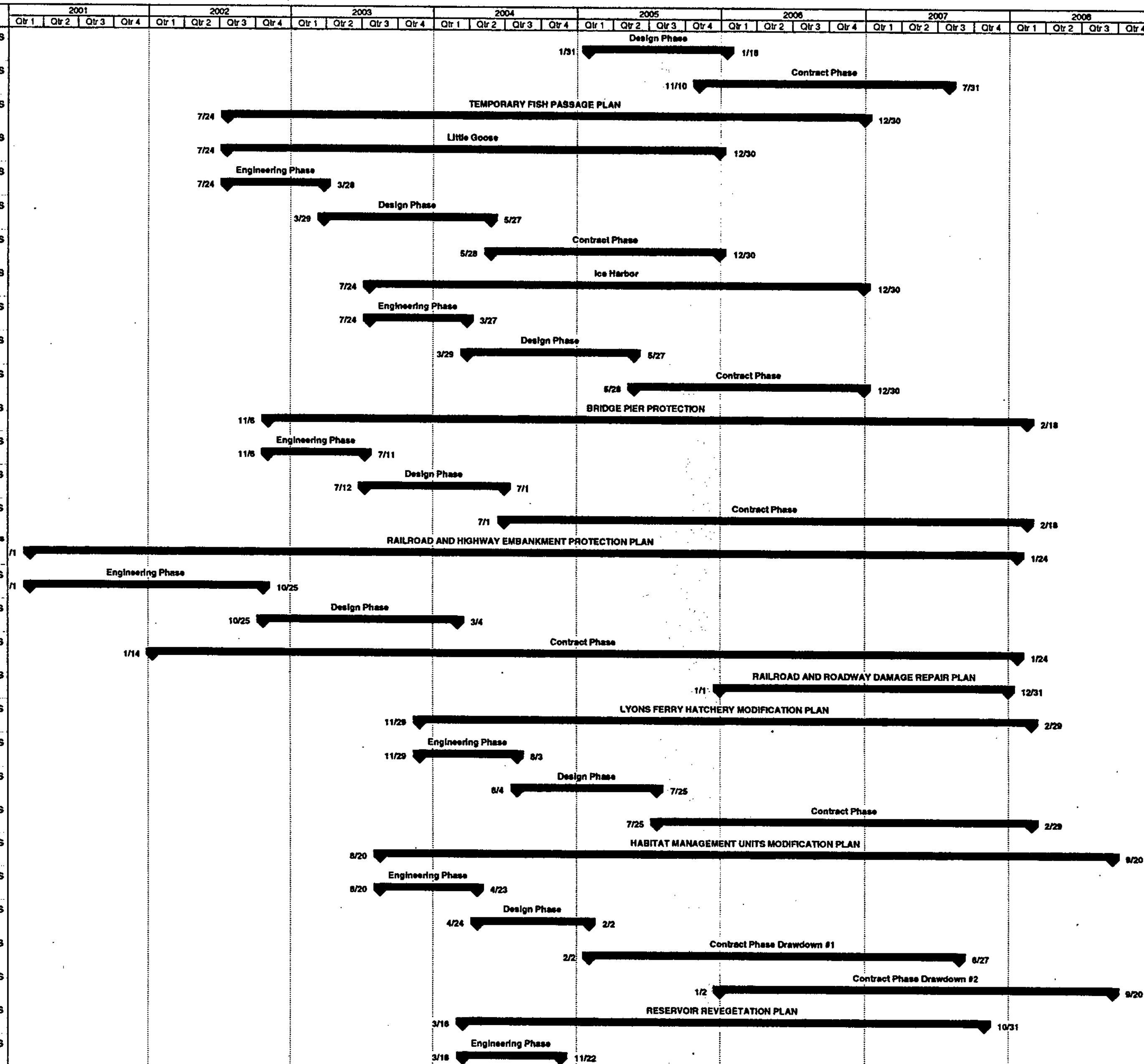


Project: Drawdown Implementation Schedule
Date: Thu 12/9/99

Task | Split | Progress | Milestone | Summary | Rolled Up Task | Rolled Up Split | Rolled Up Milestone | Rolled Up Progress | External Tasks | Project Summary

Figure 2-10
Drawdown Implementation Schedule

ID	Task Name	Duration
188	Design Phase	305.5 days
191	Contract Phase	560.5 days
221	TEMPORARY FISH PASSAGE PLAN	1414 days
222	Little Goose	1079 days
223	Engineering Phase	213 days
226	Design Phase	365 days
229	Contract Phase	501 days
235	Ice Harbor	1101 days
236	Engineering Phase	213 days
239	Design Phase	365 days
242	Contract Phase	523 days
248	BRIDGE PIER PROTECTION	1678.3 days
249	Engineering Phase	213 days
252	Design Phase	304.2 days
255	Contract Phase	1161.1 days
277	RAILROAD AND HIGHWAY EMBANKMENT PROTECTION PLAN	2185 days
278	Engineering Phase	517.1 days
283	Design Phase	425.9 days
286	Contract Phase	1911.3 days
317	RAILROAD AND ROADWAY DAMAGE REPAIR PLAN	647 days
318	LYONS FERRY HATCHERY MODIFICATION PLAN	1355.1 days
319	Engineering Phase	213 days
322	Design Phase	304.2 days
325	Contract Phase	837.4 days
336	HABITAT MANAGEMENT UNITS MODIFICATION PLAN	1618 days
337	Engineering Phase	213 days
340	Design Phase	243.4 days
343	Contract Phase Drawdown #1	826.6 days
349	Contract Phase Drawdown #2	873 days
355	RESERVOIR REVEGETATION PLAN	1157.7 days
356	Engineering Phase	213.7 days

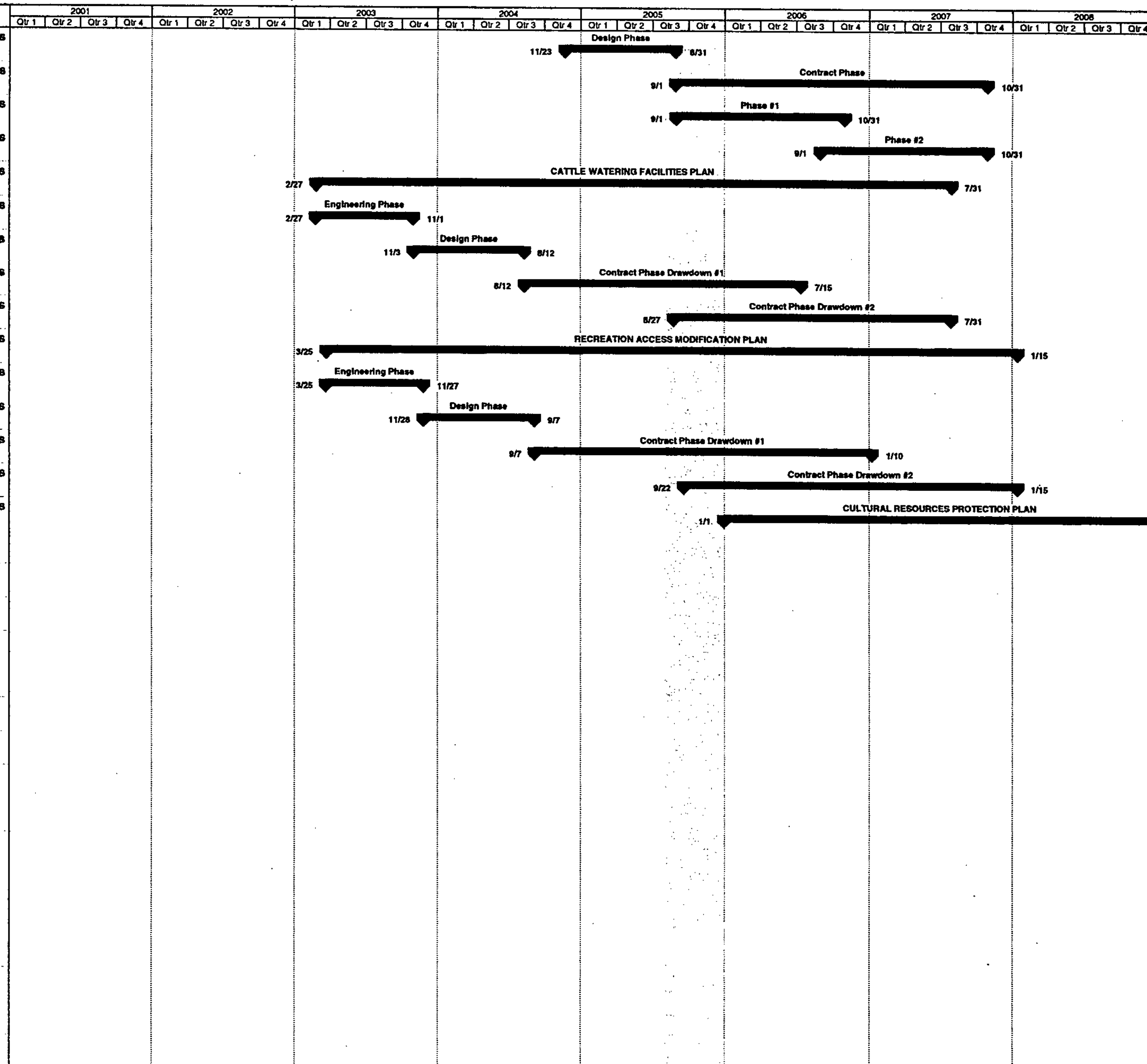


Project: Drawdown Implementation Schedule
Date: Thu 12/9/99

Task · Split · Progress · Milestone · Summary · Rolled Up Task · Rolled Up Split · Rolled Up Milestone · Rolled Up Progress · External Tasks · Project Summary

Figure 2-10
Drawdown Implementation Schedule

ID	Task Name	Duration
359	Design Phase	243.7 days
382	Contract Phase	700 days
383	Phase #1	384 days
387	Phase #2	371 days
371	CATTLE WATERING FACILITIES PLAN	1408.5 days
372	Engineering Phase	213 days
376	Design Phase	243.4 days
379	Contract Phase Drawdown #1	618.6 days
385	Contract Phase Drawdown #2	624.5 days
391	RECREATION ACCESS MODIFICATION PLAN	1531 days
392	Engineering Phase	213 days
395	Design Phase	243.4 days
398	Contract Phase Drawdown #1	757.6 days
403	Contract Phase Drawdown #2	747 days
408	CULTURAL RESOURCES PROTECTION PLAN	961 days



Project: Drawdown Implementation Schedule
Date: Thu 12/9/99

Task · Split · Progress · Milestone · Summary · Rolled Up Task · Rolled Up Split · Rolled Up Milestone · Rolled Up Progress · External Tasks · Project Summary

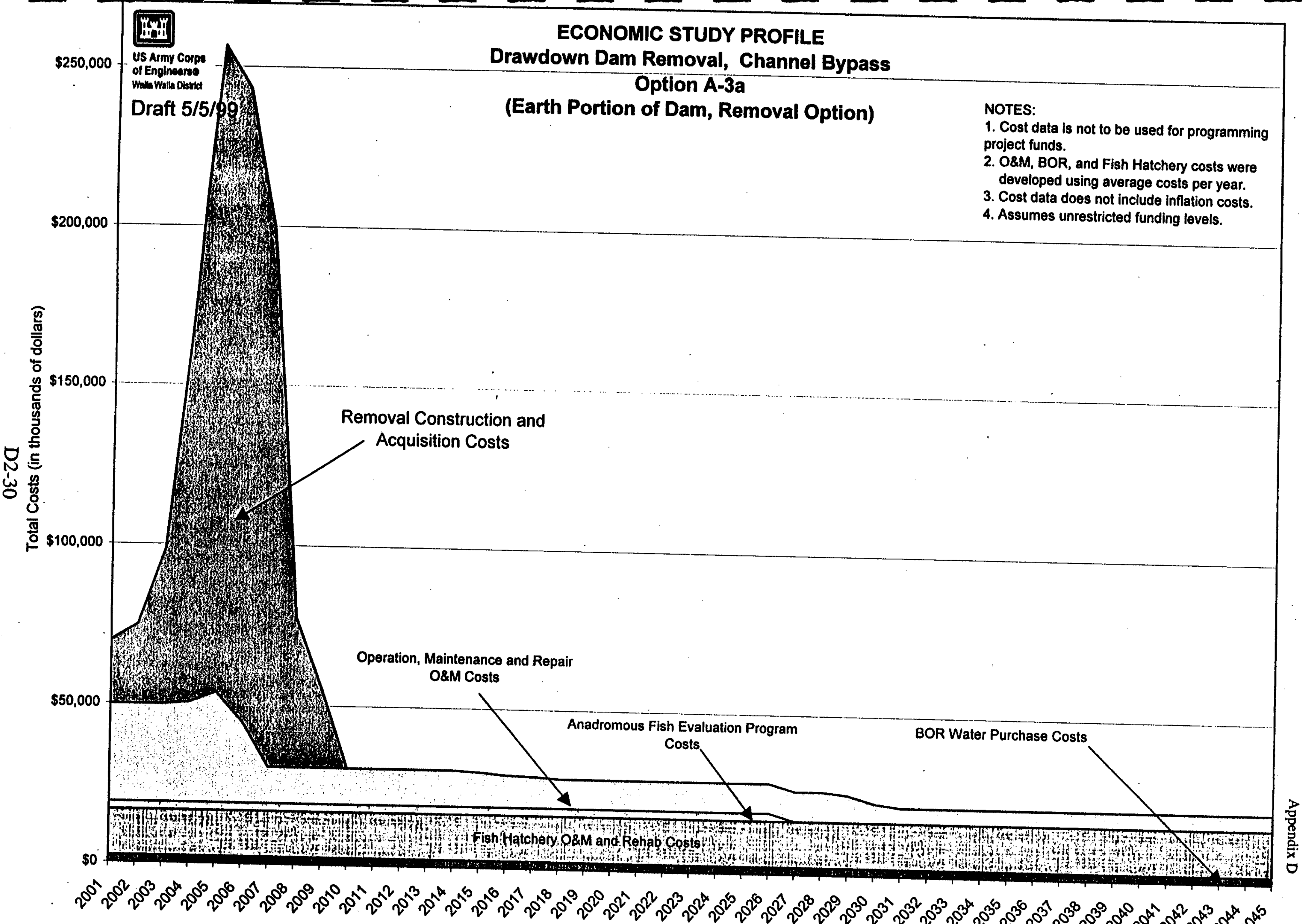


Figure 2-11. Economic Study Profile: Option A-3a, Drawdown Dam Removal, Channel Bypass

3. Critical Criteria for Concept Design

This engineering study for implementation of the Natural River Option for drawdown is based on a number of criteria, assumptions, and key considerations. Three primary goals were established. The first primary goal is to provide a system that operates in a manner that improves fish passage conditions. Structural modifications and construction operations must be structured so that fish benefit over the long term. However, some short term conditions may not provide ideal conditions for fish migration. A second primary goal is to develop modifications that provide satisfactory fish passage at a reasonable minimum cost. Those includes preventing conditions that may result in significant maintenance costs at a later date. The third primary goal is to develop modifications and the associated construction operations that are reasonable, safe, and constructable. Specific criteria resulting from the primary goals are as follows:

- Continuous Fish Passage
  Since the purpose of drawdown is to improve the passage of juvenile stocks, construction activities will be orchestrated in a manner to ensure, so far as possible, that ongoing fish passage is not adversely affected. Many in-water construction operations will be scheduled to be done during the in-water work window.

- No Catastrophic Drawdown
  The evacuation of the reservoirs will be done at a fixed rate of 0.6 meter (m) (2 feet) per day. A higher rate could cause significant slope failures in the reservoirs, putting highways and railroads out of service. Further detailed evaluation of slope materials may allow some modification of this rate. Drawdown rates may vary during the period of drawdown after considering the location of critical embankments relative to the final water surface. An erosion-based method of embankment removal was not considered a feasible option for this study for reasons discussed further in Section 4.2 of this appendix.

- Minimal Cost
  When considering various options for implementing drawdown, the lowest cost option will be a primary consideration. The goal of this study is to identify the major activities necessary to implement a four-reservoir drawdown and to document a feasible, reasonable method to accomplish those activities.

- Mitigation Measures
  This concept design includes numerous construction activities to implement a drawdown and modify existing structures in the reservoirs. Direct measures are those activities necessary to evacuate each reservoir, remove a portion of the dam structure, and establish a river channel at the project site. In addition to these activities, law or policy requires other activities. These include modifications and repairs to transportation facilities (railroads, highways, etc.) adjacent to and across the river. Legal agreements that allow cattle access to the river for watering purposes require that alternate cattle watering measures be provided. Cultural resources protection is mandated by public law.

  Several discretionary measures are also included since they are authorized under current and anticipated project authorizations. They include modifications to current wildlife lands,

modifications to an operating fish hatchery, and measures to provide river access and appropriate recreation facilities.

Measures not authorized under this project are mitigation measures for agricultural, commercial, and industrial users of water, private water wells, utility river crossings, and other commercial and private interests.

- Safety Measures
  The issue of safety will be addressed on several levels. The obvious concerns relate to safety measures implemented for each construction task. Construction activities must be planned and executed in accordance with the Corps of Engineers Safety Manual. This is a comprehensive document that details the requirements to which all construction activities must adhere.

  Safety will be addressed in a more general nature in how the design and construction activities are structured. For example, a critical element in the project design is to develop the risks associated with dam breaching and structure the work to minimize risk of catastrophic failure of the embankment and include provisions to deal with low probability flood waves. In addition, restricting access to the construction areas, the river, and the river shores is part of all construction safety measures. Examples of specific safety issues include, restrictions on boating in the construction areas and restrictions on public access to construction work areas.

4. Reservoir Evacuation Plan

4.1 General Considerations

The four Corps dams on the lower Snake River were constructed without an outlet facility that allows the evacuation of reservoir water to a minimum. Structural modifications or additions must be done to each project in order to discharge reservoir water in a controlled manner. To accomplish a controlled drawdown, the reservoirs can be drafted to near-spillway-crest by discharge over the spillways. To draft the reservoirs below this discharge elevation, a low-level outlet to provide a controlled release of water is necessary so that subsequent structural removal can be accomplished without a significant head of water against the structures. Currently water passes the structure through the powerhouse while generating power, when water flows are high or power demands low, water is discharged over the spillways. Water is also discharged over the spillways during the Spring and Summer Months for fish passage. In their current configurations, the projects are incapable of reservoir releases below spillway crest elevation.

In the SCS Phase I report, reservoir releases below spillway crest were planned through construction of a multi-stage cofferdam system that provided a low-level outlet. However, use of sheetpile systems for the anticipated heads and usage was unprecedented, and the foundation conditions might have prevented use of sheetpile systems for such an application. Unfortunately more conventional systems such as excavating a new outlet and gated structure through the embankment, abutment, or concrete structure was not practical given the large quantity of water to discharge at relatively low head.

This study investigated the feasibility of using the existing turbines and turbine passages to discharge water, determined the number of required passages to draft the necessary amount of water, and defined the modifications to the turbine and generator equipment that would be necessary. Controlling the water flows at the varying heads is critical to avoid both structural damage that could lead to a catastrophic release of water, or equally devastating, a reduction in the volume of water passed that would adversely affect fish survivability. The turbine passages must operate as outlets during the period of reservoir drawdown and continue to operate as outlets until the embankment can be fully excavated to create a new river channel.

As previously mentioned in Section 3, catastrophic release of water in the drawdown is unacceptable. This can occur when embankment removal is performed using explosives or hydraulic erosion. Once water begins to flow over the embankment, the water eroded the embankment material and very shortly removes the embankment. To breach the embankments while impounding a high head would lead to rapid embankment erosion, uncontrolled erosion, and a high rate of water level drop. Rapid drawdown rates would cause serious damage to railroads and high embankments in the reservoir. The rapid embankment erosion could also harm fish passage. Furthermore, there is significant evidence that an erosion-based method of embankment removal would not achieve complete removal of the embankment material. Uncontrolled hydraulic excavation of the material could result in an unsatisfactory channel configuration that would have obstructed access and be difficult to excavate.

The related issues of risk and contingency planning are discussed in various sections of the annexes. After more detailed designs have been established for turbine modifications and the subsequent

structures removal and structural stabilization in the reservoir, a more clear picture of the weak elements can be determined. At that time a series of analyses will be performed establishing the probabilities of certain events and failures leading to a quantification of the risk. With risk established, contingency plans can be established for many of the activities to assure that the range of possible problems do not derail the drawdown and protection process.

4.2 Period of Drawdown

Currently, the in-water work window is designated by the NMFS as December 15 through March 15 of each year. This is the period of time during the year when work activities would impact the least number of fish migrating in the river. However, this time period is not sufficient to perform all the construction activities necessary to produce the new river channel. Furthermore, considering the high probability of excessive river flows during this period, reservoir drawdown must be performed in advance of this period. Otherwise, the risk of catastrophically breaching the embankment is high.

Use of existing turbine passages with proposed modifications for drawdown provides a total discharge capacity of approximately 1,700 cubic meters per second (m^3/s) (60,000 cubic feet per second [cfs]). Snake River flows are below 1,700 m^3/s (60,000 cfs) during only a limited time period. When considering the probabilities of flows exceeding this threshold value, turbine passage usage can only occur during the months of August through December of any year. The probability of flows exceeding 1,700 m^3/s (60,000 cfs) during January are calculated to be 20 percent (termed "the 20-percent-chance annual frequency") and increase significantly through the winter and spring.

Drawdown and concurrent embankment excavation must be initiated shortly after the end of the spill season, when river flows recede to below 1,700 m^3/s (60,000 cfs), and must be completed by the end of December for each project.

4.3 Hydraulic Studies

Initial concepts for reservoir evacuation included the installation of multiple outlets in one or more powerhouse bays, replacing the existing turbines and generators. Such a modification resulting in significant loss of power production and a great potential for consequent increase in spillway discharge. The resulting high saturated gas levels in the river from increased spill can have devastating effects on fish. Alternate concepts were developed where the existing turbine passages and equipment could be utilized as discharge outlets. A number of evaluations were necessary to confirm this approach.

Turbine operating characteristics for the anticipated conditions during drawdown were determined using an operating scale model of a turbine at Voest-Alpine MCE Corporation in Linz, Austria. Hydraulic model studies were also performed at the Corps's Waterways Experiment Station (WES) in Vicksburg, Mississippi, using a 1:25 section model of a Lower Granite Dam turbine passage to establish the hydraulic characteristics of the passage when the blade had been removed from the turbine. These studies led to the conclusion that, for reservoir flows up to approximately 1,700 m^3/s (60,000 cfs), a reservoir drawdown could be accomplished using the turbines and turbine passages. For the higher head condition, up to three turbines would be operated as the reservoir dropped over a range of 6 m to 12 m (20 feet to 40 feet), which is 15 m to 21 m (50 feet to 70 feet) below normal operating range. At the lower elevations, flows would shift to the remaining three turbine passages, from which the turbine blades would have been previously removed. These three passages provide the low-head discharge.

The major modifications required for this process include installation of alternate cooling water sources and plumbing, modification to existing intake gates, construction of draft tube bulkheads to facilitate turbine operations at low water elevations, and removal of three turbine blades from the turbine units.

At this stage of the reservoir evacuation, the reservoir would be at the lowest elevation possible through turbine discharge. It is estimated that the remaining head on the embankment cofferdams would be between 4 m and 7 m (12 feet and 22 feet). This is the head differential that would be present during final excavation of the embankments.

4.4 Turbine Modification and Operation Plan

Modifications to turbines and associated equipment would be necessary to allow the use of the turbine and passages to function as outlets. Modifications must be completed well in advance of drawdown. However, some turbine capacity must be maintained during the previous spill season in order to aid in controlling the saturated gas levels in the river. Excessive spillway use raises saturated gas levels to unacceptable levels. Modifications must be scheduled so that turbine use is maximized and spillway use is limited to acceptable timeframes.

The operating turbine and generator serve to dissipate the energy of a high head and allow the passage of a significant volume of water. In order to make the turbines operate at lower heads than the current operating head, numerous modifications must be made. At each project, three turbine units must be modified to operate over a range of low head conditions. These modifications are as follows:

- Emergency Closure Devices
  Existing emergency closure devices should be in operating condition. The use of these gates is only in the event that conditions develop that could cause failure of the water outlet process and the purpose is to isolate that turbine passage. Currently, the intake gates at each project are either raised (with the hydraulic operators disconnected) or removed for improved fish passage purposes. During a reservoir drawdown, the fish screens would be removed. The intake gates should be connected to the hydraulic operators and stored in the normal position, ready for emergency use.
- Cooling Water System
  The cooling water supply system for turbines and generators must be modified to operate under a variable head condition during drawdown. There are two broad categories of water that need to be provided depending on absolute pool level and whether generation is necessary. The first category is the water required for thrust- and guide-bearing cooling, gland water, air compressors, station service transformers, and heat pumps for cooling the control and computer rooms. This water is required as long as the units are turning, whether they are generating or not. The bearing cooling water can be shut off if the units are stationary. The second category is for cooling water for the air coolers in the generating unit. This cooling water is required only if the generating units are operating. The generating unit transformers are air-cooled.
- Trash Rack Modifications
  Investigation is necessary to assure that the trashrack structures are adequate for debris loads over the range of head pressures to which they will be subject. Some strengthening has been

assumed to be necessary for drawdown conditions. The trash racks should be inspected and repaired as necessary prior to drawdown. A significant effort will be required to keep the trash racks clear of debris during drawdown.

- Draft Tube Bulkheads
  When more than one project is drawn down at once, the tailwater of the upstream project will drop significantly. For example, normal minimum tailwater at Lower Granite is 193 m (633 feet). If Little Goose reservoir is also drawn down, the tailwater at Lower Granite will fall to about 190 m (624 feet). This drop in tailwater will cause serious cavitation problems for the turbines. One way to avoid these problems is to partially lower the draft tube bulkheads to a fixed location to create an orifice in the draft tube. This would increase head losses and create an artificial tailwater for the turbines. The loading on the bulkheads and supporting structures would be in the opposite direction from how they were designed, and the forces would no longer be just static loading. A more complete structural analysis would need to be completed before implementing this action. A hydraulic evaluation is also necessary to determine the operating characteristics of these bulkhead and the related operators and controls. Each project only has one set of draft tube bulkheads, so additional bulkheads for the remaining five units would need to be purchased.

- Turbine Blade Removal
  Up to three turbines at each project will require removal of the turbine blades to operate as bladeless runners. This will allow maximum discharge of water through the turbine passages at low heads. Removal is expected to be done several months in advance of drawdown by cutting the blades and removing them through the intake slot or out through the draft tube. The alternate process of unstacking the generator and removing the turbine is too time consuming and too expensive if lost power is considered.

- Performance Instrumentation
  Instrumentation is necessary to monitor conditions of the turbine during out-of-the-ordinary operations. The instrumentation identifies developing conditions that may lead to a failure of the system and may prevent the necessary discharge of water. Early warning provided by instrumentation allows operators to react and implement contingency plans. Instrumentation should measure acceleration, shaft run out, increased leakage, bearing temperatures, structural and mechanical vertical displacement, and pressures at the head cover, intake, and draft tube man doors. There should also be instrumentation to detect runner blade impact on the discharge ring.

- Operation
  Detailed procedures would be developed to operate the turbine and generator equipment in the unusual mode. Significant advanced testing is anticipated to establish operational limits and appropriate responses to developing conditions. For example, operation below the speed no load (SNL) condition is possible, but would require direct manual operation. Such operation should only be considered after more critical evaluation. Such operation would require an operator at each turbine to adjust the wicket gates and monitor the turbine speed and other unit parameters. The turbine generators must be disconnected from the power grid by opening the breakers.

- Contingency Plans
  In the event that equipment fails to operate as expected during the reservoir evacuation,

contingency plans must be in place in order to continue the drawdown process and complete the embankment breach. Typical contingent operations might be operating turbine units manually at or below speed-no-load status, breaching the embankment cofferdams at higher heads, and/or utilizing a modified intake gate for regulated flow through the turbine passages. The development of specific contingency plans is beyond the scope of this study.

A detailed report on the Turbine Passage Modification Plan is provided in Annex A of this appendix.

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5. Dam Embankment Removal Plan

5.1 General Considerations

Each of the four lower Snake River dams is a composite dam comprised of several sections. Each consists of concrete non-overflow sections, powerhouse section, spillway section, navigation lock, and an embankment section. Figure 3 through 10 provide a plan view drawing of each project and an aerial photograph of each project.

A screening-level evaluation was made of several methods for dam breaching. The methods considered were:

Modify the spillway bays to serve as outlets by removal of ogee sections.

Modify the navigation lock to serve as an outlet by removal of the upper sill block.

Modify powerhouse turbine passages to serve as outlets.

It was soon obvious that the critical element in breaching any part of the structure was that an adequate outlet had to be provided in advance of drawdown. Impacts to navigation prevented continued consideration of measures that removed the locks from service at least 1 year in advance of reservoir drawdown. Outlet construction through the powerhouse modifications would remove turbines from service and seriously increase gas levels in the river for the 2 spill seasons preceding drawdown. Not enough time was available to make spillway modifications, since they must be done during the period between spill seasons.

The second formidable consideration was that the portion of the dam to be removed must be fully removed during the priority between spill seasons. Otherwise significant cofferdams are required to isolate the work area and provide a temporary structure that retains a reservoir at a level where spillway features and fish facilities remain operational. It was clear that excavation of the embankment was the only viable method to meet the time constraints. By contrast, removing concrete sections as a means to breach the structure would require much more preparatory activity and significantly more expense, and might require several construction seasons to accomplish the drawdown and return the river to a free flowing condition. Consequently, this study team selected dam embankment removal as the method for breaching the dams.

5.2 Schedule and Risk Constraints

As discussed in section 4.2, drawdown of the reservoirs can only occur during a period of time when river flows are less than the discharge capacity of the project. The risk of overtopping the embankment increases to unacceptable levels beyond January 1 each year Catastrophic failure of the embankment due to overtopping is unacceptable. Reservoir evacuation cannot commence until early August of any year. Embankment removal must follow reservoir evacuation. To avoid catastrophic breaching of the embankment dam, the embankment dam must be full excavated by early January of the same year.

Design and sequencing the drawdown process requires that risks be evaluated of overtopping the embankments and cofferdams during the construction period. A subsequent failure analysis is necessary to route and contain the flood wave to minimize adverse impacts. Such an analysis

provides a means to evaluate contingency plans in order to be prepared for schedule delays and extraordinary hydrologic events. Such analyses are critical to the design of this system. Selecting higher contingency costs compensates the absence of such evaluations at this stage of the project.

5.3 Geotechnical Conditions/Considerations

The existing embankment dams consist of silty, impervious core material; sand and gravel shells; filters; and slope protection comprised of rockfill and riprap. The gravel shell materials were obtained from the gravel terrace borrow sources. The gravel terraces consist of sands, gravels, cobbles, and boulders eroded and deposited by glacial outwash flows. The materials are typically rounded to subangular. The volumes of material in the embankment, cofferdams, and abutments that would be excavated from each of the four dams are summarized in Table 5-1.

Table 5-1. Embankment Excavation Quantities

	Quantity (cubic meters)				
Material	Lower Granite	Little Goose	Lower Monumental	Ice Harbor	Total
Core Material	240,200	138,300	78,300	7,500	464,300
Gravel Fill (shell), Including Rockfill and Riprap	1,101,700	978,000	675,200	59,500	2,814,400
Cofferdams	276,400	263,900			540,300
Abutments and Cofferdams			4,567,340	4,199,480	8,766,820
Total Volumes[1]	1,618,300	1,380,200	5,320,840	4,266,480	12,585,820

1/ Quantity of embankment excavation only. Does not include common excavation from the abutments at Lower Monumental and Ice Harbor.

Riprap and rockfill are present on the upstream and downstream faces of the embankment. Riprap and rockfill for original construction were obtained either from designated quarries or from required excavation for the concrete structures. Although core materials are not required for any aspects of the river channelization, the core material would be excavated and stockpiled separately for potential future use. Core materials would be saturated and would dry slowly.

5.4 Excavation Scheme

Once the elevation of the reservoir has reached an acceptable level, excavation of the embankment material would commence. Excavation would be a coordinated effort involving several excavators and supporting off-road hauling vehicles and dozers. The proposed construction schedule and construction cost estimate assume a rate of material excavation ranging from 764 cubic meters (m^3) (1,000 cubic yards [cy]) per hour for the narrow embankment sections near the top of the dam to 4,128 m^3 (5,400 cy) per hour for the wide embankment sections near the base of the dam. The rock slope protection, the rockfill, filter material, and impervious core material would be hauled to stockpile areas on the respective shores. Since reservoir evacuation initially exposes a minimum volume of embankment, full-scale excavation of the embankment dam is not expected to commence

until 4-5 weeks later. This operation, using standard high-capacity construction equipment, is expected to easily catch up and keep pace with the reservoir drawdown.

The embankment would be excavated to the foundation, approximately 5 m (15 feet) below the natural water surface elevation, leaving material in place at the upstream and downstream zones to serve as cofferdams. The Lower Granite and Little Goose embankment dams were constructed with cofferdams that were incorporated into the embankment. With minor modification, these zones could serve as cofferdams again. At lower Monumental and Ice Harbor Dams, designated zones to act as cofferdams were not previously used as cofferdams. Additional material modifications are anticipated to stabilize these embankment sections.

With the majority of the fine material removed from the river section, the cofferdams would be systematically removed using excavators and draglines. Material would be handled in two groups: 1) silt would be stockpiled apart from the sands and gravels, and 2) riprap might be stripped from the surface and utilized elsewhere if the rock size were appropriate. Stockpile locations for each project have been identified within a 3.2-kilometer (2-mile) haul distance of the excavation.

Because of the permanent upstream pool, upstream gravel and filter zones and the central core are in a saturated condition. The developed production rates for equipment groups account for problems associated with handling saturated materials. Fortunately, the gross quantity of silt core is approximately 12 percent of the total quantity excavated. Specific issues related to handling and stockpiling materials will also need to be addressed. Preparation of stockpile areas and haul roads will be done well in advance of the start of drawdown.

The volume of material to be excavated is summarized in Table D-1. The duration of embankment excavation at Lower Granite and Little Goose dams would be 28 and 21 days, respectively. Cofferdam removal requires an additional 25 days all of which to be done during and following reservoir drawdown. The duration of embankment excavation at Lower Monumental and Ice Harbor dams would be 55 and 61 days, respectively. Cofferdam removal requires an additional 20 to 30 days. The excavation of abutment and downstream channel sections would be done in advance of drawdown. The embankment configurations at Ice Harbor Dam and Lower Monumental Dam would require extensive excavation and railroad relocation to form an adequate channel for fish passage.

The initial breach of each cofferdam would be excavated with a hydraulic excavator or by dragline. The water flow through the breach would erode the silty, sandy , and gravel materials. The downstream cofferdam would be breached first. The head differential across the cofferdam would stabilize quickly and removal of the rest of the cofferdam by dragline could proceed. Breaching the upstream cofferdam would be much more dramatic. The 4- to 7-m (12- to 22-foot) head differential would result in significant flow velocities that might readily scour the embankment material in the breach section. A breach section of at least 15 m (50 feet) and up to 33 m (100 feet) is anticipated. Once the differential head had equilibrated, excavation of the remaining cofferdam would proceed. It is likely that further erosion of material might occur during excavation.

A detailed discussion of the Dam Embankment Excavation Plan is provided in Annex B of this appendix.

5.5 Temporary Fish Handling Facilities

It is critical that embankment removal be done during the time of least risk of high river flows. Generally that period is between July and March. However, early-winter runoff and winter storms narrow that safe range to August through December. This time period corresponds to the active period of adult Chinook and steelhead migration. Any construction activity during this period that "blocks" the river must include a provision for passage by these adult fish. Once drawdown begins, the existing facilities would no longer be operable, and no passage could occur for up to 120 days until a free flowing channel is established.

Several options were considered for alternate fish passage. The two options determined to be the most feasible were modifying the existing facilities to operate in an extended mode or capturing and transporting the adult fish. These are described below.

- Facility modifications would involve extending the entrances to fish ladders to the drawdown river channel and providing supplemental "attraction" water in quantity and orientation to attract adult fish to the ladder. Water pumps and appropriate modifications would be added to supply 2 m^3/s (75 cfs) to the fish ladder. An upstream fish conveyance feature must also be added to allow passing fish to slide down from the ladder to the ever lower reservoir.
- The alternate method would collect all the adult fish at Little Goose Dam during the first drawdown season and at Ice Harbor Dam during the second drawdown season. Collected fish would be transported by truck to a release point upstream of the affected area. This process would involve construction of an adequate trap facility at the two projects and the manufacture or modification of up to 26 semi-trailers for what would be, at times, around-the-clock operation.

On the recommendations of NMFS, the trap and haul options was selected. NMFS considered this method to have a higher probability of success in assuring the migration of adult fish compared to the facilitation of the in-river migration of fish. In-river conditions during the initial drawdown of the reservoirs could create significant migration problems. A detailed discussion of the Temporary Fish Passage Plan is given in Annex C of this appendix.

6. River Channelization Plan

6.1 Hydraulic Considerations

The goal of river channelization is to force the river channel around the concrete structures remaining in the natural river channel. In most cases the embankment breach is not located in the natural lay of the river. In order for the river to transition around the structures, levees must be installed. Without these levees, the flow conditions around the structure are very unpredictable and could lead to erosion that will prevent the migration of fish during certain flow conditions.

The ultimate determination on the effectiveness of levees in providing a smooth river transition and preventing erosion damage at higher flows will be based on large-scale model studies of each project. Specific details regarding levee geometry and ancillary features cannot be formulated without the use of such a tool. Detailed discussions on the character and transport of sediments are contained in Appendix F, Hydrology Appendix. Detailed discussions on the environmental and biological effects of sediments and contaminants are contained in Appendix C, Water Quality Appendix, Appendix B, Resident Fish Appendix, and Appendix A, Anadromous Fish Appendix.

6.2 Channelization Approach

Hydraulic concerns require the use of channelization levees in forming a natural river. Model studies will determine whether both upstream and downstream levees are necessary. This Feasibility Study assumes that the complete channel around the structures will be formed by a levee.

Once the embankment structure is fully removed, the river must route itself around the concrete structures that remain. In most cases the river has to dogleg back in a manner that is not consistent with the meander of the river. A channelization structure must be added to force the river into this alternate routing. The goal of this channelization is to provide a smooth transition around the structure so as not to create a condition that may impede fish passage at the full range of operational flows from 600 m^3/s to 5,000 m^3/s (20,000 cfs to 170,000 cfs). In addition, the channel must operate without damage for flows of up to 13,000 m^3/s (450,000 cfs) so that, when water flows recede, the channel remains operational for fish passage.

The channelization levees would be constructed of shotrock manufactured as a by-product of the production of bank protection rock. The levees need not be impervious (except for the option that was developed for the cost estimate in which the concrete structure is removed). They are configured to provide freeboard through the 100-year flood (10,000 m^3/s [350,000 cfs]). (Freeboard is the distance the water surface is maintained below the top of the excavation.) The levees for each site require 6,000 m^3 to 8,000 m^3 (200,000 cy to 300,000 cy) of material to be placed during the time period just following embankment removal and the start of spring runoff, November through March.

In addition to their channel function, the levees would serve as a security barrier to keep people out of the abandoned site. Security fencing will be installed on the levees and linked to the perimeter fencing to prevent entrance to the abandoned site.

6.3 Levee Design

During construction of the four lower Snake River dams, extensive hydraulic model studies were performed on various cofferdam configurations. For Lower Granite, Little Goose, and Ice Harbor dams, the cofferdam configurations were not much different than the proposed levee configurations. Those model studies provide a reasonable basis for comparison to the numerical evaluations of this Feasibility Study.

Construction methods used during construction of the cofferdams are similar to those assumed for the levees in this study. The approach is simply to dump material from trucks into the river to form a levee section and advance this section from the shore. Placement of slope protection and any other special features would follow.

There is no need to dewater the area within the levee. As noted earlier, the concrete structures would be abandoned. Ground contours just upstream and downstream of each powerhouse show very deep excavations to facilitate intake and discharge for the powerhouse. These excavations would remain deep pools. If water flow were blocked to the interior of the levee, the water quality in this area might deterioration to undesirable conditions. Consequently, the levees would be constructed to be permeable. The levee material will allow the passage of water through the levee, creating a slow flow condition within the levee. Passage of water through the structure will facilitate the complete change of water within the levee.

Construction-era cofferdams were primarily natural sands and gravels excavated from borrow sources in and adjacent to the future reservoir. Silts used for the impervious core of these cofferdams was not as common and required significant processing was done to separate it from sands and gravels. The sand-gravel composition of the cofferdam was satisfactory for a temporary facility where constant monitoring and repair capability was present. That configuration is unsatisfactory for long-term levees subject to flow velocities that can be quite damaging. The levees, therefore, will be constructed of shotrock, that is angular basalt rock ranging up to 300 millimeters (mm) (12 inches) in diameter. This material is a byproduct of riprap production necessary for railroad and highway embankment protection (see Annex F for more details). Processing of the several million cubic meters of waste from the riprap production should net sufficient material for levee construction. The required quantity of shotrock will be barged to each project site in advance of drawdown and stockpiled for later use in constructing the levees.

6.4 Fish Passage Features

Under ideal circumstances, the breach in the embankment dams would be sufficient to allow fish passage, as river velocities with flows up to approximately 170,000 cfs would not impede the upstream migration of fish. This appears to be the case for Lower Granite and Little Goose Dams. The velocity conditions through the embankment dam breaches at Lower Monumental and Ice Harbor are higher. Model studies, should design efforts proceed to the next stage, will provide more detailed information on velocity and flow direction for more specific locations. Such conditions will necessitate the construction of in-water features to aid fish in migrating through the high velocity reaches.

Criteria for fish passage through this new channel were developed based on published information of appropriate velocity-reach data for various species of fish (FFHA, 1990), (Corps, 1991). Table 6-1 provides the criteria used for locating fish passage features. Simply stated, channel velocities below 1.5 meters per second (m/s) (5 feet per second [ft/s]) require no supplemental fish passage

features. Channel velocities above 1.5 m/s (5 ft/s) require features in the river to produce rest areas. The higher the velocity, the more numerous and frequent the rest areas.

Normal river features that provide rest areas are eddies and rocks. A range of options are available to provide these rest areas for this new channel. They range from boulders strategically placed along the river to anchored boulders, anchored concrete features, and a full-scale fish ladder. The selection of which feature is most appropriate will also depend on model studies of how each feature functions over the range of river flows.

For this study, it was assumed that a series of precast concrete units, anchored to the river bottom, would provide suitable resting points in the channel and would withstand the forces generated by the range of river flows.

Table 6-1. Required Spacing of Fish Passage Features

Velocity of Flow (m/s)	Velocity of Flow (ft/s)	Sustained Distance (m)	Sustained Distance (ft)
0.6	2		Indefinite
0.9	3		Indefinite
1.2	4		Indefinite
1.5	5		Indefinite
1.8	6	61	200
2.1	7	37	120
2.4	8	24	80
2.7	9	15	50
3.0	10	9	30
3.4	11	6	20
3.7	12	3	10

For flows where velocities do not exceed 5 ft/s, no fish passage enhancements are necessary. As flow velocities increase from 5 fps to 12 fps, the addition of fish passage enhancements must be done. The spacing of these features ranges from about 200 feet at 5 fps to 10 feet for 12 fps. Model studies will provide the final verification of performance.

This criterion presumes that velocities of flow approaching and exiting the new channel are less than velocities in the channel. The existing concrete structures would create a flow constriction in the channel that increases the velocity. However, if velocities along the bank are more than 1.5 m/s (5 ft/s), channel enhancement features such as anchored boulders that provide eddies and pools could be added along the bank to provide flow discontinuities.

The maximum flow against which adult fish are assumed to swim upstream is 4,813 m^3/s (170,000 cfs), which is approximately the flow within the 2-year exceedence probability.

For more details concerning the River Channelization Plan, see Annex D of this appendix.

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7. Other Implementation Plan Modifications

7.1 General Considerations

A wide range of planned modifications and mitigative actions are necessary as a result of drawdown and are integral parts of the drawdown implementation plan. Modifications to stabilize and repair the infrastructure are addressed in the subsequent sections titled Bridge Pier Protection, Railroad and Highway Embankment Protection, Drainage Structure Protection, and Road and Railroad Damage Repair. Wildlife and habitat issues are addressed under sections on the Lyons Ferry Hatchery Modifications, Habitat Management Unit Modifications, Revegetation, and Cattle Watering Facilities. Issues regarding Recreation Access and Cultural Resource Protection are discussed in sections on their respective plans, as follows.

7.2 Bridge Pier Protection

Nine highway or railroad bridges cross the reservoirs and up to 15 bridges cross tributary rivers to the reservoir. Drawdown of the reservoirs would result in high velocity river flows at most of these bridge sites. Because of scour that might result from the high velocity flows, modifications would be required to stabilize the bridge piers and abutments. This study concluded that, based on the boundaries of the natural river and the condition of the existing piers, 25 bridge piers would require stabilization by rock placement or sheetpile installations, in addition to back protection at each of the bridges.

In general, the modification process would be to access each bridge by barge and install a sheetpile ring around each designated pier. This would be followed by placement of the appropriate fill material within the cells and on the abutment slopes. Final cutting of sheetpile and placement of concrete would be completed after drawdown.

For more details on the Bridge Pier Protection Plan, see Annex E of this appendix.

7.3 Railroad and Highway Embankment Protection

More than 156 kilometers (79 miles) of railroad and highway adjoin the existing reservoirs. Many of these thoroughfares were relocated to their existing location as part of constructing the lower Snake River hydropower facilities. In some cases, drawdown of the reservoir will not directly impact the function of the road or rail beds, and no modifications are required. The only sections of concern are those sections constructed on engineered fills that will be in direct contact with the natural river. Of the 145 kilometers (90 miles) of railroad and highway embankment which are on engineered fill sections, 71 kilometers (44 miles) would be exposed to river flows at the new lower river elevations. Preliminary assessments indicate that the exposed slopes must be protected with properly sized rock to prevent slope erosion and undermining of the rail or road beds. Because access to many of these locations is difficult and access to some would be impossible after drawdown, the rock needed for the embankments would be transported by barge in advance of drawdown.

Basalt rock for riprap will be processed at eight quarry locations. Extensive reconnaissance and exploration would be necessary to establish a viable rock source for the size and quantity of required

rock. See Annex F for details and locations. Rock from each site would be used to service an appropriate reach of the river to minimize transportation costs.

Processed rock would be hauled from the quarry to a barge loading area and loaded onto rock barges for transportation. The rock would be transported to several underwater stockpile locations. Stockpile sites have been tentatively located by evaluating the geomorphology data presented in Appendix H, Fluvial Geomorphology. Sites were identified where pre-drawdown and post-drawdown water velocity conditions, water depth, and substrate conditions were least detrimental to migrating and spawning fish. After drawdown of the reservoirs, access to the stockpile locations and bank protection sites would be made using existing roadways and construction-era roadways exposed by drawdown. Nearly 1,000,000 cy of rock would be needed to provide the required bank protection.

A detailed discussion of the Railroad and Highway Embankment Protection Plan is contained in Annex F.

7.4 Drainage Structures Protection

Concurrent with the efforts to protect embankments, protection would need to be provided for nearly 5400 drainage structures that run through these embankments. These structures were designed to allow passage of water from existing upslope drains through highway and railroad embankments into the reservoirs created by the dams. A majority of the drainage structures require protection on the slope so that discharge water does not erode the embankment. Such protection would be placed during the placement of bank protection rock. Modifications to the existing drainage structures would offer some logistical challenges. Because the drains are spaced far apart, have difficult land access, and require placement of narrow strips of riprap extending down steep slopes, it is not practical to treat these slopes in advance of drawdown. Access to rock and each site would be made using existing roadways and construction-era roadways exposed by drawdown. See Annex G of this appendix for more details regarding the Drainage Structures Protection Plan.

7.5 Railroad and Roadway Damage Repair

In addition to the modifications that are needed in advance of drawdown for certain railroad and roadway embankments, as described earlier, some of these embankments will undoubtedly also be damaged during the drawdown itself. As drawdown occurs, areas of the embankments along the river are anticipated to fail due to steep slopes, saturated soils, and pore pressure increase. The location and extent of embankment failures is extremely difficult to predict based on the uncertainty and variability of materials used in constructing the embankments. Consequently, the study team determined that a prudent estimate of damage could be done based on the recorded embankment damage data from the 1992 drawdown of Lower Granite Reservoir.

Most embankment repairs cannot be performed until drawdown is accomplished. Also, in some areas, it may be necessary to wait several weeks after drawdown to allow the materials to drain and stabilize before repairs can be initiated. It is anticipated that most repairs will be completed within a few months after drawdown is complete.

A complete discussion of the Railroad and Roadway Damage Repair Plan is provided in Annex H of this appendix.

7.6 Lyons Ferry Hatchery Modifications

The Lyons Ferry Hatchery, at the confluence of the Palouse and Snake rivers, was constructed to serve as mitigation for fish and wildlife habitat lost or altered by construction of the four lower Snake River dams. Fish raised at the Lyons Ferry Hatchery include steelhead, Chinook salmon, and trout for release into the Snake River and its tributaries. This study initially assumed that hatchery operations cannot be fully interrupted by drawdown. This is based on a NMFS determination that fish hatchery operations will continue in the region during and after drawdown. This is due, in part, to the determination that recovery of fish stocks will not be immediate, but will take several years.

A total of eight water wells, located adjacent to the reservoir 3.2 kilometers (2 miles) upstream of the hatchery, supply over 114 m^3/min (30,000 gallons per minute [gpm]) of process water for hatchery operations. The water is transported 2,966 m (9,730 feet) via a 1,524-mm (60-inch) diameter concrete cylinder pipe (CCP), 1,920 m (6,300 feet) of which is submerged in the reservoir and supported by 104 pipe pile bents. When the reservoir is drawn down, the wells may not produce the required quantity of water. New wells, drilled after drawdown, will be needed to compensate for this deficit. The underwater pipe supports are not sufficient to support the pipe. Additional pipe bents must be added to stabilize the pipe sections since installing a new pipeline along an alternate pipe routing in advance of drawdown is not feasible. Numerous other modifications are necessary to maintain an operational hatchery. These details are presented in Annex I of this appendix.

7.7 Habitat Management Unit Modifications

There are designated lands along the reservoirs that provide protected wildlife habitat areas to replace lands lost because of the reservoirs. Operation of these areas will continue until such time as the natural habitat develops and eliminates the need for these managed units. There are over 30 habitat management units (HMUs) on the river with nine of them intensively managed. Intensive management means primarily that irrigation systems have been installed to water certain lands. Modifications resulting from the drawdown would include re-fencing lands to prevent access to HMUs and reconfiguring irrigation systems to operate under new water surface conditions.

The complete HMU Modifications Plan is presented in Annex J of this appendix.

7.8 Reservoir Revegetation

As the water surface drops, up to 8,000 hectares (20,000 acres) of land will be exposed. Development of native plants on these lands would need to be encouraged and the growth of undesirable plants would need to be discouraged. The timing of drawdown makes planting and seed germination challenging. A systemic aerial application of seed and fertilizer is planned following the drawdown. Additional seeding and willow and cottonwood plantings are planned for subsequent seasons. See Annex K for more details concerning the Reservoir Revegetation Plan.

7.9 Cattle Watering Facilities Modifications

Original land use agreements allowed cattle ranchers access to the reservoir for water for their cattle. In order to honor these pre-existing agreements, new cattle watering facilities would need to be developed. As many as 60 cattle corridors currently exist on the river. To avoid possible damage by cattle to habitat and spawning areas along the new river, cattle corridors would be grouped where possible and wells would be installed that would provide water via solar powered pumps to stock

watering tanks. Fencing would be installed to form a barrier to cattle access to the river banks. The modifications must be done following drawdown when the groundwater conditions have stabilized.

Details of the Cattle Watering Facilities Plan are shown in Annex L of this appendix.

7.10 Recreation Access Modifications

A system of 33 recreational facilities provide numerous sites for camping, boating, moorage, day use, and hiking on the affected reach of the river. While there is no doubt that recreation pursuits will continue after drawdown, the nature of recreation may shift. A plan has been developed to determine what modifications would be necessary to the existing sites based on current assumptions concerning recreational use.

Marinas would be eliminated from the recreation sites. While camping and motor camping would continue, some sites would no longer maintain river access. It is anticipated that 11 sites would be completely abandoned and demolished. Fifteen sites would be modified to discontinue certain activities. At 15 sites, boat ramps or other features would be relocated to better access the new river. New recreation facilities to better meet the evolving opportunities on this river system are beyond the scope of this implementation plan and were not studied.

For more details regarding the Recreation Access Modification Plan, see Annex M of this appendix.

7.11 Cultural Resources Protection

Over 300 known cultural resource sites exist along the river. Several legal and regulatory mandates exist that require protection of these and other identified sites. (See Cultural Resources Appendix N). The sites include villages, campsites, cemeteries, and rock shelters. A range of treatments has been developed depending on the degree of protection and the public accessibility to the site. The Cultural Resources Protection Plan developed for the drawdown would offer protection measures for these sites, as detailed in Annex N of this appendix.

8. Non-Federal Modifications

The majority of the issues discussed in this appendix are modifications necessary to implement a river drawdown or modifications necessary as a result of a river drawdown. Those modifications are considered an integral part of the drawdown implementation plan and were included as part of the projects funding requirements.

There are significant other impacts resulting from a river drawdown that are necessary to maintain certain commercial operations or private enterprise, but were not included as part of the projects funding requirements. Several of those are discussed in this section because they either represent a significant engineering and construction effort or are modifications that are similar in nature and scope to modifications that are part of the implementation plan. If a Natural River alternative is selected, Congress may or may not choose to fund these non-federal modifications. Estimated costs are shown in Table 8-1.

Table 8-1. Non-Federal Modifications Summary of Costs, in $1,000

Project	Direct Costs	Contingency	Escalation	Total
Ice Harbor Project				
Irrigation System	$224,216	$67,264	$54,693	$346,174
Groundwater wells	$9,188	$9,185	$3,450	$21,823
Lower Monumental				
Groundwater wells	$6,233	$6,228	$2,339	$14,800
PGE Gasline Crossing	$5,916	$2,071	$1,573	$9,560
Little Goose				
Groundwater wells	$3,901	$3,896	$1,461	$9,258
Lower Granite				
Groundwater wells	$8,909	$8,906	$3,346	$21,161
Private water users	$551	$166	$133	$851
Potlatch water intake and effluent diffuser	$7,912	$2,772	$2,091	$12,775

Notes:

1. Direct costs include lands, administration, engineering, construction management.
2. Contingency percentage is specific to each item.
3. Escalation is to mid-point of construction.
4. Private water users are Atlas Sand and Rock, Lewiston Country Club, and Clarkston Municipal Golf Course.

8.1 Irrigation System Modification Plan

There are eight active large-scale pumping plants in the 21-kilometer (13-mile) reach of the Snake River upstream of Ice Harbor Dam. They supply irrigation water for circle irrigation systems, vineyards, orchards, pulp trees, and numerous row crops. Water is required between the months of

February and October. The peak demand for water supplied by these pumping plants currently totals 19 m^3/s (680 cfs). This peak demand is required for a sustained period of 2 to 3 months, depending on the weather conditions.

A modified water supply system would be required for irrigation following drawdown. Modifications to each plant were considered, but rejected. They included relocating intakes, adding sedimentation ponds, modifying and replacing pumps. The shallow depth of the natural river, the heavy sediment loads, and the 4.6-to-6.1-m (15-to-20-foot) fluctuation in river level made individual modifications unreliable. A corporate system using a single intake structure and a pressure pipeline was selected instead. Only one iteration of concept development was done. More comprehensive design requires that the system be reconfigured for the actual number of viable water users weighing the specific requirements of individual users.

A 19-m^3/s (680-cfs) water intake would be sited in the narrow river section upstream of the irrigated lands. This narrow, self-scouring reach of the river maintains a water depth of over 12 m (40 feet) during low flow conditions under natural river flows of 600 m^3/s (20,000 cfs). This intake would consist of five bays with the appropriate trashracks and fish screens. Pumps configured for approximately 30,000 horsepower would be required to supply 19 m^3/s (680 cfs) at flow rates of 2 m/s (8 ft/s) through the piping system at the appropriate pressures. The piping system would consist of 13 miles of pipe with mainline diameters ranging from 3.7 m to 2.1 m (12 feet to 7 feet). At two locations, 1,066-mm (42-inch) branches would cross the river to provide water to the existing pumping stations on the north shore. The pipeline would interface with individual distribution systems via manifold systems to booster pump stations.

The presence of heavy sediment load in the river water remains a major problem. One alternative is to pump water to a 202-hectare (500-acre) reservoir to provide some silt separation and surge protection for the system. The water is subsequently pumped into the pressure pipe distribution system.

The work would be done in advance of the irrigation season preceding drawdown. Two to 4 years of advance notice may be needed to complete all necessary tasks.

Complete details on the Irrigation System Modification Plan are provided in Annex O of this appendix.

8.2 Water Well Modifications

Water wells existing along the lower Snake River supply domestic water, agricultural water, and some commercial uses. The study team assumed that most of the commercial use of water other than for agriculture is supplied by municipal water systems. These water wells range from shallow wells collecting water from surface sources to deep wells drawing from the deep basalt formations. Drawdown of the water surface in the four reservoirs ranges from a change of only a few feet at the upstream end of the reservoirs to as much as 24 m (80 feet) upstream of each dam site. The aquifers adjacent to the river could be greatly affected by the change in water surface. The degree of impact will depend, in part, on the geologic formation supplying the water to the well, the proximity of the well to the river, and the depth of the well. While it is not possible to characterize each well along the affected river reach, in general the most adverse effect from drawdown will be to wells drawing water from the shallow aquifers.

An inventory of the existing water wells within approximately one mile of the Snake River was developed from information presented on the logs of the drilled wells. The well logs were obtained from the records of the Washington Department of Ecology (Ecology) office at Spokane, Washington. There are approximately 180 recorded water wells in the designated study area. A detailed evaluation of each well was not done. The response of the aquifers to variations in water surface is a complex relationship and far beyond the scope of this overview. Instead, it was more prudent to evaluate a representative sample of the 180 recorded wells and proportionately apply those modifications to the whole. For each of those sample wells, modifications included increasing the depth of the well below the estimated new groundwater surface and installing a new pump and associated hardware to pump against the increased head.

The well modifications must be done following drawdown when the groundwater conditions have stabilized. Measures to provide temporary water during drawdown were not investigated.

Details of the Water Well Modification Plan are presented in Annex P.

8.3 Water Intakes

The Potlatch Corporation in Lewiston, Idaho, manufactures and supplies wood, paper, and consumer products. The primary plant water intake is located on the Clearwater River in the Lower Granite Reservoir. The intake has a peak capacity of 1,500 m^3/s (52,000 cfs). The lower water surface elevation resulting from drawdown of the reservoir is expected to be too low during low flow periods to allow this intake to function properly. To address this issue, the study team proposed installing auxiliary intakes in deep water to supply the existing wet well. Four screened inlets constructed within sheetpile enclosures would be used. See Annex Q of this appendix for more details concerning the Potlatch Corporation Water Intake Modification Plan.

Several other private water intakes exist along the Lower Granite Reservoir. Atlas Sand and Rock maintain an intake for water supply for rock crushing and concrete operations south of Lewiston along the Snake River. The cities of Clarkston, Washington, and Lewiston, Idaho, each operate a water intake to supply irrigation water to golf courses. Trailer-mounted pumps with flexible intakes and appropriate connections and controls are proposed to restore the capability of these users to access surface water from the Snake River. See Annex R of this appendix for more details regarding Other Water Intakes Modification Plan.

8.4 Wastewater Effluent Diffusers

Potlatch Corporation discharges effluent to the river. Treated effluent from the plant is conveyed from the plant through a buried pipeline to an in-water diffuser near the confluence of the Clearwater River with the Snake River. Drawdown of the Lower Granite Reservoir would expose the top of the polyethylene diffuser. The proposed modification is to relocate the diffuser to a deeper reach of the river downstream from its current location. Various reaches of new pipeline and diffuser would be installed using sheetpile sections to dewater the work areas. Other measures to treat effluent water currently under evaluation are not included in this study. See Annex S of this appendix for more details concerning the Potlatch Corporation Effluent Diffuser Modification Plan.

8.5 Utility River Crossings

The only utility crossing the Snake or Clearwater rivers necessitating modification is the Pacific Gas and Electric (PG&E) natural gas transmission line that crosses the Snake River near Lyons Ferry

State Park. One 914-mm (36-inch) gas line was installed across the river in the 1950s, and a second line was installed in the 1980s. Replacement of the gas lines is necessary since scour conditions may damage the existing pipe. Installation would occur after drawdown of the reservoir using sheetpile systems to enclose and dewater the work area. In addition to new concrete-encased pipe sections, costs are estimated for stabilizing adjacent banks and abandoning the existing pipe. See Annex T of this appendix for more details concerning the PG&E Gas Transmission Main Crossings Modification Plan.

9. Hydropower Facilities Decommissioning

9.1 General Considerations

The process of lowering the reservoirs and breaching the dam embankments would eliminate navigation and hydropower, two significant uses of the four lower Snake River dams. After breaching of the embankments, the remaining dam structures would consist of a navigation lock, a powerhouse, spillway, concrete and embankment non-overflow dams, fish facilities, and other support facilities. The study team for decommissioning these projects considered two major actions:

- Breaching the embankment dam and, by constructing levees, permanently channeling the river around the remaining dam structures, and leaving the dam structures in place.
- Breaching the embankment dam, temporarily channeling the river around the remaining dam structures, and removing the dam structures from the river.

The term decommissioning as used in the FR/EIS refers to removing structures and equipment from service. The term deauthorizatoin as used in this FR/EIS refers to a congressional action to eliminate the purpose or mandate for existence of the project. In the case of a drawdown, both may be utilized.

Each major action leads to multiple sub-options for the treatment of the remaining structures and equipment. Some of those options hinge on the effect that reservoir and dam removal would have on the recovery of endangered and threatened fish stocks.

The first action requires the disposal of most of the project equipment and on-site waste materials, but leaves the structures in place, yet off-limits, to public access. Under this first action, two approaches to disposing of project facilities and equipment were identified:

- The Abandon Option is to cease all activity, dispose of all equipment and materials, and abandon the site. If, at some later date, fish stock recovery is determined to be a success because of the new river configuration, the abandoned structures could continue to remain in the river in perpetuity or be removed from the river at that time.
- The Mothball Option is to suspend all operations and maintain equipment in working condition until a decision is made to abandon or restore operations. If it is concluded that reservoir and dam removal have no effect on the recovery of the listed fish stocks, it would be possible to restore the projects to an operating condition.

In the second major action, removal of the dam structures from the river is not reversible. Removal of the dam structures would include: actions to dewater the demolition site, significant explosive and impact demolition, excavation, and transportation of equipment and waste materials to designated waste and salvage areas.

Although the study team concluded that leaving the concrete dam facilities in the river would be the major action selected for this implementation plan, the team did develop a concept for demolition and removal of the existing dam structures. Both actions are discussed in more detail below. For a

detailed discussion of the Hydropower Facilities Decommissioning Plan, see Annex U of this appendix.

9.2 Decommissioning while Leaving the Dam Structures in Place

9.2.1 Disposal Options

As mentioned earlier, the two disposal options considered by the study team are the Mothball Option and the Abandon Option.

The purpose of the Mothball Option is to protect and preserve the existing equipment so that the equipment can be restored to operating condition at a later date, or to at least maintain the option for future restoration until such time as that decision can be made. The scope and costs of such operations were based on current maintenance requirements for the Lower Granite Dam.

The Abandon Option would involve ceasing all operations, removing all salvageable equipment, and securing the structure from public access. Only minor maintenance activities would be performed to maintain project lighting and site security.

The four lower Snake River hydropower facilities range in age from 23 to 48 years old. It is clear from the maintenance records that the older facilities are exhibiting problems associated with aging equipment. Much of the equipment is at the extreme end of its useful life and would likely require replacement during a project restart. It would not be economical to maintain the equipment for 20 years and then have to replace it if the hydropower project is restarted.

Furthermore, the cost of removing and relocating equipment, considering its age, is excessive. Much of the equipment is customized for its current location and would require modification for use by other Federal projects. The study team concluded that, as a whole, there is no economic salvage value for the equipment at each of the plants. Consequently, this implementation plan is based on abandoning the dam facilities.

The Abandon Option requirements associated with decommissioning will be performed during and after drawdown. The only item that needs to be completed before drawdown is the construction associated with providing power from the public utility. Power for lighting and security will be needed when power production is stopped at the facility.

9.2.2 Disposition of Industrial Waste

Each project contains numerous materials or items that can be classified as hazardous/dangerous materials, substances, chemicals, or wastes under Federal and state laws. When the projects are decommissioned, all items that are designated as solid wastes will need to be identified, characterized, and disposed of in accordance with Federal, state, and local regulations and codes. A detailed summary of identified materials and disposition is contained in Annex U of this appendix.

9.3 Removing and Disposing of the Concrete Structures

The abandoned structures consist primarily of mass concrete for the navigation locks, spillways, powerhouses, and non-overflow sections. Other structures that would be abandoned include embankment sections, steel structures, and numerous support facilities on the site. Only a cursory effort to develop demolition quantities was undertaken in the development of this concept.

A large volume of concrete exists below the elevation of the river bottom. For this concept, it was assumed that concrete removal would be done to an elevation two meters below river bottom. This means approximately 40 percent of the structures would be removed. The concrete rubble would be placed along the old river bank. Steel structures and debris would be hauled to waste areas or salvage areas as necessary. This work would be performed during the year following breaching of the embankment.

Full removal of the concrete structure would require construction of an impervious cofferdam/levee around the demolition site that would be subsequently removed. The levees in this approach must be able to prevent much of the water from leaking through or under the cofferdam. Permanent levees are not required since, after removal of the structures, the river can flow on its natural alignment. However, channelization would be required during the time that the concrete structures are being demolished and removed.

Construction-era cofferdams included a cutoff trench with impervious fill. The construction process involved end-dumping sandy gravels to form the cofferdam section. Once the section was complete, a trench was excavated in the center of the section using a bentonite slurry to hold the trench open. This trench was subsequently filled with a thick formulation of sandy silt and water and allowed to displace the bentonite slurry. This fill made the cofferdams relatively impervious. After dewatering the interior of the cofferdams, any resulting leakage was collected and pumped back to the river.

The same cofferdam construction method would be used for the drawdown if concrete structures were to be removed. The shotrock would not be used for these cofferdams. Local sources of sand and gravels that are readily available at each dam site would be used instead. Silt materials might be easier to collect since deposition has occurred over the past 20 to 40 years.

A detailed description of the Concrete Structures Removal Plan is provided in Annex V.

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10. Implementation Schedule

10.1 General

The general process for implementing the work is to perform a three-step process consisting of 1) preparation of a detailed design report, 2) preparation of contract documents, and 3) performance of construction.

The detailed design report, formerly designated a General Design Memorandum or a Feature Design Memorandum, details the process of identifying, evaluating, and selecting a design option. The activities often are precluded by a survey of each construction site to establish the land configuration. Subsurface explorations using intrusive methods such as drilling, excavating, and sampling and/or geophysical methods such a pulse-velocity, radar, or other subsurface logging methods are conducted at this stage. For some features, hydraulic models must be constructed and flow conditions evaluated for a range of flow and physical conditions. Options are developed for the feature and detailed evaluations are made to select the most favorable option. The selected option is often further developed so that a reliable schedule and cost estimate may be generated.

After review and approval of the detailed design report, preparation of the plans and specifications can proceed. This phase requires completion of the feature design and the development of contract documents. The documents must be prepared in a manner that allows bidders to prepare a realistic bid proposal, that presents features in manner that is constructable, and that provides implementation and operations that address the relevant environmental concerns.

Once a contract has been awarded, the construction can begin. The short-term nature of many of the tasks coupled with the complexity of implementation will require the participation of many individuals and organizations. Construction activity spans a time period of approximately eight to nine years. During the peak years, expenditures are estimated at $200 million in a single year. The bulk of the work is performed during a 3-month period. Extensive contractor participation is necessary for this level of effort. Significant administration and construction management participation is also required.

The schedules in Annex W reflect reasonable time durations to perform these efforts. They identify time for producing detailed design reports, contract documents, peer and policy reviews, advertising periods, and construction operations.

10.2 Overall Implementation Schedule

The implementation of drawdown may be grouped into three distinct phases. The preparatory phase includes the work necessary to be done in advance of drawdown in order to be able to perform drawdown and the work necessary to continue operations during drawdown. The drawdown phase is the work required during and immediately following drawdown of the reservoirs. Numerous tasks are anticipated that will need to be performed following drawdown. The period of time that all these occur is shown in Annex W.

A key decision in implementing drawdown is the sequence of dam breaching. There are many options ranging from concurrent breaching of all four dams in a single construction season to

individual breaching of each dam during different seasons, with many combinations within this range.

Breaching individual dams on different years greatly simplifies construction operations and focuses attention on one project at a time. The first project provides a troubleshooting opportunity so that subsequent projects can be breached more effectively. Events that may lead to delays that prevent breaching during the designated season are more effectively controlled increasing the likelihood of on-schedule completion. Funding is less difficult to secure because annual requirements can be spread out over a longer period of time.

Breaching of an embankment structure will generate the migration of embankment silts and sands down river. A much more significant effect is the migration of silt deposits and higher velocity river flows that erode those deposits. Silts suspended in the water may be at very high concentrations during the drawdown period of August to December and possibly higher levels during the high flow months of January through June. The effect of this silt and sediment is expected to have a serious negative effect on adult fish migration and a lesser effect on juvenile migration.

If the four dams are breached simultaneously, then this condition will be concentrated to the shortest time period thereby minimizing the negative effects on migrating fish. Biologists expect that expanding this situation as long as four consecutive years could be detrimental to the species (Jones, 1999). Breaching the four dams over two consecutive years provides for realistic implementation of all the construction activity. This two season breaching is considered less devastating than other options that require longer time periods.

An aggressive schedule to simultaneously breach four dams needs much more detailed evaluation. An evaluation of risks and impacts of specific construction activities is necessary to produce a plan that contains the appropriate backup plans and contingencies to guarantee that the work can be completed in the short timeframe. At the current level of study, the study team has determined that that too many factors may go wrong that may force the project into a 2-year breach schedule. Until those uncertainties can be resolved, a 1-year breach schedule cannot be considered.

Figure 12 summarizes the implementation schedule for the major work items for a breach plan where two dams are breached during one construction season and the remaining two dams are breached the following construction season. For more detailed implementation schedules, see Annex W of this appendix.

11. Implementation Cost Estimate

11.1 General

The study team developed construction costs from the engineering concept-level designs for this Feasibility Study. The costs are based on the scope of work, assumptions, and methodology presented in the engineering annexes (Annexes A through V of this appendix). Estimates were completed for two options for returning the lower Snake River to natural flow conditions: 1) removing the earthen embankment dam then channeling the river around the remaining concrete structure, and 2) full removal of the earthen and concrete dam structures.

Conceptual design report and supporting documents to identify the estimated construction costs of modifications required to bring the lower Snake River back to natural stream flow conditions were prepared by the Corps and a number of supporting organizations. Two separate reports, titled *Embankment Excavation River Channelization and Removal of Concrete Structures* (Raytheon, 1998) and *Lower Snake River Reservoir Stabilization Plan* (Raytheon, 1997), document the assumptions and quantities used in the estimates for the construction efforts involving the reservoirs, dams and locks. Three conceptual design reports concerning the installation of natural gas river crossings (TDH, 1998c) and the modification of water intake (TDH, 1998a) and effluent diffuse (TDH, 1998c) facilities for Potlatch Corporation were prepared by Thomas, Dean, and Hoskins, Inc. of Lewiston, Idaho. The Corps' Walla Walla District developed concepts and quantities for the remaining mitigation and modification projects. Details regarding assumptions, project design concepts, and quantities prepared by the various contractors and the Corps are documented in Annexes A through V of this appendix.

The level of detail for design and subsequent development of costs is at the concept-level. Price levels are for October 1998. The construction costs were developed using cost estimating software. Subsequent summary spreadsheets add engineering, construction management, administration, and contingency costs. Administrative and management costs are estimated at 1.5 % of the construction cost. Engineering costs for development of detailed design documents and subsequent contract documents are estimate at 8.3%. Environmental compliance is estimated at 1%. Large design costs for aerial, land, and underwater surveys, foundations and materials explorations, and hydraulic model studies were estimated separately and included with the appropriate design task. Construction management costs and engineering during construction are estimated at 9%. Cost escalation due to inflation assumes that project activity begins at the start of the calendar year 2000 and is projected to the mid-point of construction of each major task. The mid-points range from 2003 for the early engineering activities to 2008 for the last options.

11.2 Methodology

A feasibility-level cost estimate was developed for each of the two options. The cost estimates include costs for construction, real estate, cultural resources, engineering and design, construction management, and project management. Construction costs were prepared using the Micro Computer-Aided Cost Engineering System (MCACES) software. The estimate is based on a work breakdown structure (WBS) that was developed to seven levels, as follows: project, feature, subfeature, element, bid item, assemblies, and detail.

The major assumptions used in preparing the estimate are as follows:

- Drawdown of the reservoirs and breaching of the dams will occur at a rate of two dams per year.
- Fish passage around the projects will be maintained during construction.
- The Lyons Ferry Fish Hatchery will remain operating as near to current capacity as possible.
- The rock sources identified will have enough material available.
- In-water work will be allowed to occur during normal fish window closures. Some in-water work must occur outside the normal fish window closures.

Other assumptions are documented in the detailed estimate.

The environmental fish windows normally regulate the construction periods on the lower Snake River. This study team assumed that these requirements would have to be waived in order for this project to go forward. To accomplish the embankment breach construction, the excavation of the embankment dams must start by mid to late August (during minimum river flow), so that it will be completed by December/January. This would minimize the danger of high flows overtopping the partially removed embankments and completed levees. Bridge stabilization activities and rock stockpiling tasks must be done outside the normal work windows. Bank stabilization following drawdown must take place outside the work windows to be completed in a reasonable time period.

Because of the deadline to complete work prior to increased river flows, overtime is required for portions of this estimate. Specifically, it is required for production, transportation, and placement of rock and riprap; excavation of the embankments; placement of the levees; and adult fish collection and transportation. Work hours for these tasks were assumed to be two 10-hour shifts per day, five to six days a week.

Access to the sites was also considered. The locations of most work tasks could be accessed via county and state roads. The exceptions are the tasks to stabilize the embankments, re-vegetate the reservoirs and protect the cultural resources. Remote sites can be accessed via unimproved roads with off-highway vehicles, or by boat or helicopter.

Sand and gravel required for the various construction efforts is assumed to be available within one mile of each dam site. Rock and riprap are assumed to be quarried from a number of sites located along the Snake River. Quarries for overland haul of riprap and materials are available along the Lower Granite Reservoir. Quarries for barge hauling materials are proposed at river kilometers 35, 98 and 214 (river miles 22, 61, and 133). Disposal areas are assumed to be within one mile of the dam locations.

The estimates are based on use of common equipment and standard construction techniques. Equipment is assumed to be available on the West Coast and is reflected in the mobilization/demobilization costs. A sufficient labor force is assumed to be available in the Tri-Cities, Washington, region and the Lewiston, Idaho/Clarkston, Washington area.

11.3 Basis of Estimate

Costs are based on a typical contract bidding process with some supply contracts. The estimate assumes contracts would be awarded separately for each dam, and one contract would be awarded for procurement and placement of rock. More efficient contract combinations may be possible when work tasks are better developed. The determination of the number of contracts will ultimately depend on the schedule of work and the cost effectiveness of contract combinations.

Markups (Field Office overhead, Home Office overhead, profit and bond) were applied to the proposed prime contractors and subcontractors. Rates used were based on historical averages for similar-sized jobs.

Estimate documents include contingency and present escalation to the midpoint of construction. A contingency analysis was performed by a team of personnel knowledgeable about each phase of the project and the risks and uncertainties involved. Escalation was calculated to reflect the cost of inflation using the *Civil Works Construction Cost Index System* (CWCCIS), EM1110-2-1304.

The estimate uses Davis-Bacon Labor Rates from general decision WA980001, Modification 9, dated August 28, 1998.

Equipment rates are from *Construction Equipment Ownership and Operating Expense Schedule* EP 1110-1-8, Volume 8, September 1997.

Material pricing was obtained from vendor quotes, supply catalogs, and the MCACES Unit Price Book 96/97.

11.4 Contingency Analysis

Contingencies were developed by the study team for each task based on the team's analysis of the risk factors and uncertainties involved and in accordance with the contingency guidance provided in ER 1110-2-130-2, *Civil Works Cost Engineering.*

Annex X of this appendix lists the contingencies determined for each task and the rationale for that determination. The weighted-average contingency value for drawdown is 35 percent and for the recommended implementation actions.

11.5 Project Cost Summary

Annex X of this appendix provides a summary of the drawdown implementation costs. The costs are summarized by project and by task. The total cost of the recommended implementation action is $992 million. This cost includes required monitoring activities, operation and maintenance costs, and other related costs.

Previous estimates of cost have ranged from a high of approximately $5 billion to a low of approximately $600 million. The high cost features of earlier concepts have been eliminated and replaced with features more appropriate considering the available construction methods. The previous low estimates were revised as more details were developed for stabilization, modification, or mitigation measures.

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12. Summary and Conclusions

1. Reservoir drawdown and embankment removal must be done during the time period between spill seasons. Spillway discharge to pass the spring freshet and to aid in juvenile migration ends on approximately 1 August. River flows are below 60,000 cfs and remain low until 31 December. The probability of flows in excess of 60,000 increases significantly after 1 January. Powerhouse discharge cannot be relied upon to be the sole means of flow passage.
2. Breaching of each dam must be done by removal of the embankment section of the dam. Removal of other sections requires more time than available.
3. Embankment removal can be done with conventional equipment. The quantity of equipment anticipated for this work is not extraordinary and is not impossible to secure.
4. A key element to making this drawdown concept feasible is the use of existing turbines and passages for primary reservoir discharge. Modifications are required for this equipment to operated under the unusual low-head conditions.
5. In order to be prepared for reservoir drawdown, the turbine modifications must be done in advance of drawdown. This requires some of the turbine units to be out of service during the previous spill season. The result is that up to 3 units per project may be unavailable for during part of the spill season and will result in higher saturated gas levels in the river.
6. The physical effects of migrating sediment may have a negative impact of water intake systems in the river.
7. Fish passage is unaffected just prior to drawdown. After drawdown, fish passage will be through the new breach section of each project. During the 90-120 day drawdown, adult fish will be collected and transported around the construction and sediment-rich areas.
8. A major task is the production of rock for riprap bank protection of the railroad and highway embankments that border the river. Approximately 45 miles of shoreline requires rock placement. Over 1 million cubic yards of riprap must be produced, barge transported, and stockpiled prior to drawdown. Underwater stockpile locations have been identified in areas that are considered poor spawning areas under current conditions and will be accessible and above the water surface after drawdown. Rock production and placement requires continuous operation for up to 3 years prior to drawdown.
9. Several in-water construction activities must be done during non-work window periods. These include the stabilization of bridge piers and the placement of riprap on banks.
10. The implementation schedule requires 9 years to implement drawdown. Physical drawdown of Lower Granite and Little Goose reservoirs would occur in year 5 and physical drawdown of Lower Monumental and Ice Harbor reservoirs would occur in year 6. It is unlikely that a more accelerated schedule can be implemented.

11. The cost of the implementation is approximately $1 billion. Approximately 60% of these costs are for modifications in the reservoirs.

12. A number of modifications have been identified that are not currently considered federal costs. They are included in this study in order to estimate the costs for economic evaluations. Only Congress can authorize project funding for these items. They include private irrigation systems, private water wells and water intakes, private effluent diffusers and utility crossings.

13. References

40 CFR 261. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 261, "Identification and Listing of Hazardous Waste," as amended.

40 CFR 273. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 273, "Standards for Universal Waste Management," as amended.

40 CFR 279. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 279, "Standards for the Management of Used Oil," as amended.

40 CFR 403. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 403, "General Pretreatment Regulation for Existing and New Sources of Pollution," as amended.

40 CFR 503. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 503, "Standards for the Use or Disposal of Sewage Sludge," as amended.

40 CFR 761. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 761, "Polychlorinated Biphenyls (PCBs) Manufacturing, Processing, Distribution in Commerce, and Use Prohibitions," as amended.

40 Code of Federal Regulations (CFR) 61. Title 40, "Protection of the Environment," *Code of Federal Regulations*, Part 61, "National Emission Standards for Hazardous Air Pollutants," as amended.

49 CFR 192. Title 49, Transportation," *Code of Federal Regulations*, Part 192, "Transportation Of Natural And Other Gas By Pipeline: Minimum Federal Safety Standards," as amended.

50 CFR. Title 50, "Wildlife and Fisheries," *Code of Federal Regulations*, as amended.

ANSI/NFPA (American National Standards Institute/National Fire Protection Association). "Standard for the Storage and Handling of Liquefied Petroleum," American National Standards Institute/National Fire Protection Association 58 and 59. New York, New York.

Corps (U.S. Army Corps of Engineers). 1991. *Fisheries Handbook of Engineering Requirements and Biological Criteria,* U.S. Army Corps of Engineers, Washington, DC, date.

Corps. 1993. *1992 Reservoir Drawdown Test, Lower Granite and Little Goose Dams*. U.S. Army Corps of Engineers, Walla Walla, District. Walla Walla, Washington. December 1993.

Corps. 1994. *Columbia River Salmon Migration Analysis, System Configuration Study, Phase I.* U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. April 1994.

Corps. 1996a. *System Configuration Study, Phase II, Lower Snake River Juvenile Salmon Migration Feasibility Study, Interim Status Report*. U.S. Army Corps of Engineers, Walla Walla, District. Walla Walla, Washington. December 1996.

Corps. 1996b. EM 385-1-1, US Army *Corps of Engineers Safety and Health Requirements Manual.* U.S. Army Corps of Engineers, Washington, D.C., September 3, 1996.

Davis-Bacon Labor Rates from general decision WA980001, Modification 13, dated August 1, 1999.

EM1110-2-1304, *Civil Works Construction Cost Index System* (CWCCIS), U.S. Army Corps of Engineers, Washington D.C., September 1998

EP 1110-1-8, *Construction Equipment Ownership and Operating Expense Schedule*, Volume 8, September 1997

ER 1110-2-1302, Civil Works Cost Engineering, U.S. Army Corps of Engineers, Washington D.C., 31 March 1994. ER 1110-2-13-2, U.S. Army Corps of Engineers, Washington D.C., April 29, 1994

ER 5-7-1 Appendix 2-B, U.S. Army Corps of Engineers, Washington D.C., September 30, 1992

FHA (Federal Highway Administration), 1990. *Fish Passage Through Culverts*, Report No. FHWA-FL-90-006. U.S. Department of Transportation, Federal Highway Administration, Washington, DC. November, 1990.

Jones, Richard L. 1999. *Effects of Sediments on Physiology and Migration of Resident and Anadromous Fish as a Result of Removing the Lower Snake River Dams*, Corps of Engineers Information Paper, May 1999.

NMFS (National Marine Fisheries Service). March 2, 1995. *Reinitiation of Consultation on 1994-1998 Operation of the Federal Columbia River Power System and Juvenile Transportation Program in 1995 and Future Years*. National Marine Fisheries Service. Washington, D.C. March 2, 1995.

NPPC (Northwest Power Planning Council). 1991. *Fish and Wildlife Program Amendments (Phase Two)* Portland, Oregon. December 1991.

Raytheon. 1996. *Lower Granite Dam Turbine Passage Evaluation DO-4*. Final Letter Report for U.S. Army Corps of Engineers, Walla Walla District, by Raytheon Infrastructure Services Incorporated. Bellevue, Washington. January 4, 1996.

Raytheon. 1997. *Lower Snake River Reservoir Stabilization Plan,* Volumes 1-4. Raytheon Infrastructure, Inc., for the U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. November 19, 1997.

Raytheon. 1998. *Embankment Excavation, River Channelization, and Removal of Concrete Structures*, Volumes 1-2. Raytheon Infrastructure, Inc., for the U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. February 1998.

TDH (Thomas, Dean, and Hoskins). 1998a. *Snake River Drawdown Feasibility Study for: Primary Plant Water Intake – Potlatch Corporation*. Thomas, Dean & Hoskins, Inc., for the U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. October 16, 1998.

TDH. 1998b. *Snake River Drawdown Feasibility Study for: Plant Effluent Diffuser – Potlatch Corporation*. Thomas, Dean & Hoskins, Inc., for the U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. October 16, 1998.

TDH. 1998c. *Snake River Drawdown Feasibility Study for: Gas Transmission Main Crossings – PG&E Gas Transmission.* Thomas, Dean & Hoskins, Inc., for the U.S. Army Corps of Engineers, Walla Walla District. Walla Walla, Washington. October 16, 1998.

TRACES Unit Price Book 96/97, Tri-Services Automated Cost Engineering System, U.S. Army Corps of Engineers, Engineering and Support Center, Huntsville, AL 27 March 1998

WAC 173-303. "Dangerous Waste Regulations." *Washington Administrative Code*, as amended.

WSDOT (Washington State Dept. of Transportation). September 1990. *Evaluating Scour at Bridges, Hydraulic Engineering,* Circular No. 18, Publication No. FHWA. U.S. Department of Transportation, Federal Highway Administration. Washington, D.C.

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14. Glossary

Alternative 1—Existing Conditions: The existing hydrosystem operations under the National Marine Fisheries Service's 1995 and 1998 Biological Opinions. The Corps would continue to increase spill and manipulate spring and summer river flows as much as possible to assist juvenile salmon and steelhead migration. Juvenile salmon and steelhead would continue to pass the dams through the turbines, over spillways, or through the fish bypass systems. Transportation of juvenile fish via barge or truck would continue at its current level.

Alternative 2—Maximum Transport of Juvenile Salmon: The existing hydrosystem operations plus maximum transport of juvenile salmon, without surface bypass collectors. The number of juvenile fish transported via barge or truck would be increased to the maximum extent possible.

Alternative 3—Major System Improvements: The existing hydrosystem operations and maximum transport of juvenile salmon, but with additional major system improvements (such as surface bypass collectors) that could be accomplished without dam breaching.

Alternative 4—Dam Breaching: Natural river drawdown of the four lower Snake River reservoirs.

Anadromous fish: Fish, such as salmon or steelhead trout, that hatch in fresh water, migrate to and mature in the ocean, and return to fresh water as adults to spawn.

Bulkhead channel: Channel through which fish are carried upward through the turbines via a bulkhead slot if they are not diverted by turbine intake screens.

Bypass channel: Fish diverted from turbine passage are directed through a bypass channel to a holding area for release or loading onto juvenile fish transportation barges or trucks.

Collection channel: Holding area within the powerhouse that fish enter after exiting the bulkhead slot.

Cultural resources: Archaeological and historical sites, historic architecture and engineering, and traditional cultural properties.

Dam breaching: In the context of this FR/EIS, dam breaching involves removal of the earthen embankment section at Lower Granite and Little Goose, and formation of a channel around Lower Monumental and Ice Harbor.

Dissolved gas supersaturation: Caused when water passing through a dam's spillway carries trapped air deep into the waters of the plunge pool, increasing pressure and causing the air to dissolve into the water. Deep in the pool, the water is "supersaturated" with dissolved gas compared to the conditions at the water's surface.

Drawdown Regional Economic Workgroup (DREW): A group of regional economists studying the economic issues associated with alt4erntaive actions on the lower Snake River.

Drawdown: In the context of this FR/EIS, drawdown means returning the lower Snake River to its natural, free-flowing condition via dam breaching.

Endangered species: A native species found by the Secretary of the Interior to be threatened with extinction.

Federal Columbia River Power System: Official term for the 14 Federal dams on the Columbia and Snake rivers.

Fish collection/handling facility: Holding area where juvenile salmon and steelhead are separated from adult fish and debris by a separator and then passed to holding ponds or raceways until they are loaded onto juvenile fish transportation barges or trucks.

Flow augmentation: Increasing river flows above levels that would occur under normal operation by releasing more water from storage reservoirs upstream.

Foraging habitat: Areas where wildlife search for food.

Gas bubble disease or trauma: Condition caused when dissolved gas in supersaturated water comes out of solution and equilibrates with atmospheric conditions, forming bubbles within the tissues of aquatic organisms. This condition can kill or harm fish.

Habitat management units (HMUs): 62 parcels of land scattered along the river and reservoirs that the Corps purchased and manages as mitigation for the land that was inundated as a result of the dams and reservoirs. These HMUs are managed to replace hunting, fishing, and recreation opportunities lost as a result of inundation as well as to benefit and provide for wildlife that lost habitat to inundation.

Hydrographs: A graphic representation of stage, flow, velocity, or other characteristics of water at a given point and time.

Hydrology: The science dealing with the continuous cycle of evapotranspiration, precipitation, and runoff.

Inundation: The covering of pre-existing land and structures by water.

Irrigation: Artificial application of water to usually dry land for agricultural use.

Juvenile fish transportation system: System of barges and trucks used to transport juvenile salmon and steelhead from the lower Snake River or McNary Dam to below Bonneville Dam for release back into the river; alternative to in-river migration.

Lock: A chambered structure on a waterway closed off with gates for the purpose of raising or lowering the water level within the lock chamber so ships can move from one elevation to another along the waterway.

Lower Snake River Hydropower Project: The four hydropower facilities operated by the Corps on the lower Snake River: Lower Granite, Little Goose, Lower Monumental, and Ice Harbor.

Megawatt (MW): One million watts, a measure of electrical power or generating capacity. A megawatt will typically serve about 1,000 people. The Dalles Dam produces an average of about 1,000 megawatts.

Minimum operating pool (MOP): The bottom one foot of the operating range for each reservoir. The reservoirs normally have a 3-foot to 5-foot operating range.

Mitigation: To moderate or compensate for an impact or effect.

National Environmental Policy Act (NEPA): An act, passed by Congress in 1969, that declared a national policy to encourage productive harmony between humans and their environment, to promote efforts that will prevent or eliminate damage to the environment and the biosphere, to stimulate the health and welfare of humans, to enrich the understanding of the ecological systems and natural resources important to the nation, and to establish a Council on Environmental Quality. This act requires the preparation of environmental impact statements for Federal actions that are determined to be of major significance.

National Historic Preservation Act (NHPA) of 1966: Established federal government policy and programs on historic preservation, including the creation of the National Register of Historic Places through which the policy is implemented.

Navigation: Method of transporting commodities via waterways; usually refers to transportation on regulated waterways via a system of dams and locks.

pH: An index of the hydrogen ion concentration in water, measured on a scale of 0 to 14. A value of 7 indicates a neutral condition, values less than 7 indicate acidic conditions, and values greater than 7 indicate alkaline conditions.

Piping: Soil erosion process in which the pore pressure increases cause a vertical type fracture in the soil; this process can be a precursor to larger mass wasting failures.

Plan for Analyzing and Testing Hypotheses (PATH): A work group of regional fisheries biologists that measure projected salmon and steelhead survival rates associated with alternative actions.

Pumping stations: Facilities that draw water through intake screens in the reservoir and pump the water uphill to corresponding distribution systems for irrigation and other purposes.

Recovery: The process by which the ecosystem is restored so it can support self-sustaining and self-regulating populations of listed species as persistent members of the native biotic community. This process results in improvement in the status of a species to the point at which listing is no longer appropriate under the ESA.

Reservoir fluctuation area: Area between the minimum and maximum pool levels of a reservoir which includes the littoral, wave-action, and inundation zones.

Resident fish: Fish species that reside in fresh water throughout their lifecycle.

Riparian: Ecosystem that lies adjacent to streams or rivers and is influenced by the stream and its associated groundwater.

Rule curves: Water levels, represented graphically as curves, that guide reservoir operations. See critical rule curves, energy content curves, and flood control rule curves.

Run-of-river: This describes hydropower facilities that do not have storage or the associated flood control capacity; run-of-river facilities essentially pass through as much water as they have coming in, either through the turbines or over the spillways.

Scouring: concentrated erosive action, especially be stream or river water, as on the outside curve of a bend.

Simulated Wells Intake (SWI): Modified turbine intake that draws water from below the surface so that the surface is calmer and juvenile fish are less influenced by turbine flows. This allows juvenile fish more opportunity to discover and enter the SBC.

Slumping: A landslide; the separation of a land or soil mass from a land surface and its movement downslope.

Spawning: The reproductive process for aquatic organisms which involves producing or depositing eggs or discharging sperm.

Spill: Water released through the dam spillways, rather than through the turbines. Involuntary spill occurs when reservoirs are full and flows exceed the capacity of the powerhouse or power output needs. Voluntary spill is one method used to pass juvenile fish without danger of turbine passage.

Spillway flow deflectors (flip lips): Structures that limit the plunge depth of water over the dam spillway, producing a less forceful, more horizontal spill. These structures reduce the amount of dissolved gas trapped in the spilled water.

Surface bypass collection (SBC) system: System designed to divert fish at the surface before they have to dive and encounter the existing turbine intake screens. SBCs direct the juvenile fish into the forebay, where they are passed downstream either through the dam spillway or via the juvenile fish transportation system of barges and trucks.

Surface erosion: Movement of soil particles down or across a slope, as a result to gravity and a moving medium such as rain or wind. The transport of sediment depends on the steepness of the slope, the texture and cohesion of the soil particles, the activity of rainsplash, sheetwash, gullying, dry ravel processes, and the presence of buffers.

Surficial deposits: Unconsolidated alluvial, residual, or glacial deposits overlying bedrock or occurring on or near the surface of the earth.

Survival: The species' persistence beyond the conditions leading to its endangerment, with sufficient resilience to allow for potential recovery from endangerment. The condition in which a species continues to exist into the future while retaining the potential for recovery.

Terracing: Creation of a relatively level bench or step-like surface, breaking the continuity of a slope.

Threatened species: A native species likely to become endangered within the foreseeable future.

Total suspended sediment (TSS): The portion of the sediment load suspended in the water column. The grain size of suspended sediment is usually less than one millimeter in diameter (clays and silts). High TSS concentrations can adversely affect primary food production and fish feeding efficiency. Extremely high TSS concentrations can impair other biological functions such as respiration and reproduction.

Turbidity: An indicator of the amount of sediment suspended in water. It refers to the amount of light scattered or absorbed by a fluid. In streams or rivers, turbidity is affected by suspended particles of silts and clays, and also by organic compounds like plankton and microorganisms. Turbidity is measured in nephelometric turbidity units.

Turbine intake screens: Standard-length traveling fish screens or extended-length submerged bar screens that are lowered into the turbine bulkhead slots to divert fish from the turbine intake.

Turbine intakes: Water intakes for each generating unit at a hydropower facility.

Wetland: An ecosystem in which groundwater saturates the surface layer of soil during a portion of the growing season, often in the absence of surface water. This water remains at or near the surface of the soil layer long enough to induce the development of characteristic vegetative, physical, and chemical conditions.

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15. Annex Titles

Annex A: Turbine Passage Modification Plan
Annex B: Dam Embankment Excavation Plan
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Annex E: Bridge Pier Protection Plan
Annex F: Railroad and Highway Embankment Protection Plan
Annex G: Drainage Structures Protection Plan
Annex H: Railroad and Roadway Damage Repair Plan
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Annex K: Reservoir Revegetation Plan
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Annex M: Recreation Access Modification Plan
Annex N: Cultural Resources Protection Plan
Annex O: Irrigation Systems Modification Plan
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Annex T: PG&E Gas Transmission Main Crossings Modification Plan
Annex U: Hydropower Facilities Decommissioning Plan
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16. List of Annex Figures and Tables

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Annex A

Turbine Passage Modification Plan

Annex A: Turbine Passage Modification Plan

A.1 General

This annex describes the plans for modifying and operating the turbines to conduct a controlled drawdown of the reservoirs below spillway crest elevation.

The spillways can be used to lower the reservoir water surfaces to near the spillway crest elevation at each dam. Below the spillway crest, there are no low-level outlets except the turbine passages through the powerhouse (see Figure A1). The four lower Snake River dams were designed as run-of-river projects, meaning the turbines were designed to operate over a narrow range of forebay water surface elevations, with typically only a 0.9 meter (m)-to-1.5 m (3-to-5-foot) difference between the minimum and maximum operating pool. A report prepared by Raytheon Infrastructure Services in 1996, entitled *Lower Granite Dam, Turbine Passage Evaluation*, indicated that it may be feasible to use the turbine passages to draft the reservoirs far below normal operating range (Raytheon 1996). However, that report recommended further studies to evaluate the issues involved in such an action.

As part of this Feasibility Study, Voest-Alpine MCE Corporation in Linz, Austria, was contracted to evaluate intermediate- and low-head turbine operation using a 1:25-scale model for a Lower Granite Dam turbine. At the same time, the Corps's Waterways Experiment Station (WES) in Vicksburg, Mississippi, also conducted tests using a bladeless runner in a 1:25-scale sectional model of a Lower Granite Dam turbine. The Corps's Hydroelectric Design Center (HDC) in Portland, Oregon, provided assistance interpreting the model test data and recommending actions necessary to prepare for a reservoir drawdown using the turbine passages to draft the reservoir.

A.2 Voest-Alpine MCE Operating Turbine Model Testing

Voest-Alpine MCE performed hydraulic turbine model testing using a 1:25 scale model of the Unit 4 turbine at Lower Granite Dam. The turbine modeling was performed using Froudian methods of similitude to predict turbine operating characteristics and limits for anticipated drawdown conditions. These conditions consisted of turbine operation at and below the existing spillway crest elevation of 208 m (681 feet) mean sea level (msl) and tailwater elevations of 193 m (633 feet) msl and 190 m (624 feet) msl. The turbine model operated over head ranges of 15 m to 2 m (48 feet to 8 feet) for the tailwater of 193 m (633 feet) msl and 17 m to 5 m (57 feet to 17 feet) for the tailwater of 190 m (624 feet) msl. The testing investigated four wicket gate openings (100 percent, 75 percent, 50 percent, and 25 percent) and two runner blade angles (minimum opening of 20 degrees and maximum opening of 32 degrees). In all, approximately 160 test conditions were performed. The lower bound for the testing was established as the point where the actual turbine produces no electrical power, which is referred to as speed no load (SNL).

A.2.1 Assumptions

The study team made the following assumptions in using the turbine model results to predict actual turbine performance:

- Modeling using Froude techniques provides quantitative information regarding turbine performance.

- Model performance represents performance of actual turbine units.

- The operation of the turbine during drawdown will be a one-time operation. Operation may continue until structural safety of the unit is compromised.

- The modeling was performed on a model of Lower Granite Unit 4. Units 1-3 (Baldwin-Lima-Hamilton design) operate similarly to Units 4-6 (Allis-Chalmers design).

A.2.2 Results

Tables A1 through A4 contain data from the turbine model testing. Figures A2 through A5 present this data graphically. The model testing results, brought to actual conditions through hydraulic affinity laws, indicate the limits of actual turbine operation to be as follows:

- Pool elevation range is from 207.6 m (681.0 feet) to 196.4 m (644.2 feet) msl.
- Gross head range is from 17.4 m to 6.2 m (57.0 feet to 20.2 feet).
- Flow range is from 587 m^3/s to 218 m^3/s (20,750 cfs to 7,700 cfs).
- Wicket gate operating range is 100 percent to 38 percent, depending on the specific site hydraulic conditions.

Table A1. Model Turbine Performance at Minimum Runner Blade Angle (20 degrees) and Tailwater Elevation of 633 feet msl

Pool Elevation (feet msl)	Gross Head (feet)	Flow (cfs)	Wicket Gate Opening (%)	Power (horsepower)	Operational Feasibility
681.0	48.0	14,550	100	56,600	Yes
681.0	48.0	12,500	75	44,500	Yes
681.0	48.0	9,150	50	10,000	Yes
681.0	48.0	8,400	43	0	Yes (SNL @ Min. Gate)
675.5	42.5	8,850	50	0	Yes (SNL)
670.0	37.0	14,000	100	37,000	Yes
670.0	37.0	11,850	75	21,700	Yes
660.0	27.0	13,350	100	14,000	Yes
660.0	27.0	11,200	75	200	Yes
659.4	26.4	11,150	75	0	Yes (SNL)
653.2	20.2	12,850	100	0	Yes (SNL)

Table A2. Model Turbine Performance at Minimum Runner Blade Angle (20 degrees) and Tailwater Elevation of 624 feet msl

Pool Elevation (feet msl)	Gross Head (feet)	Flow (cfs)	Wicket Gate Opening (%)	Power (horsepower)	Operational Feasibility
681.0	57.0	15,000	100	74,300	Yes
681.0	57.0	13000	75	64,000	Yes
681.0	57.0	9650	50	27150	Yes
681.0	57.0	7,700	38	0	Yes (SNL @ Min. Gate)
670.0	46.0	14,450	100	52,700	Yes (SNL)
670.0	46.0	12,400	75	41,150	Yes
670.0	46.0	9,100	50	6,900	Yes
666.5	42.5	8,850	50	0	Yes (SNL)
660.0	36.0	13,800	100	34,550	Yes
660.0	36.0	11,700	75	20,300	Yes (SNL)
650.5	26.5	11,150	75	0	Marginal (SNL)
650.0	26.0	13,200	100	11,200	Marginal
644.2	20.2	12,850	100	0	Marginal (SNL)

Table A3. Model Turbine Performance at Maximum Runner Blade Angle (32 degrees) and Tailwater Elevation of 633 feet msl

Pool Elevation (feet msl)	Gross Head (feet)	Flow (cfs)	Wicket Gate Opening (%)	Power (horsepower)	Operational Feasibility
681.0	48.0	20,750	100	69,200	Yes
681.0	48.0	16,000	75	21,000	Yes
681.0	48.0	14,200	65	0	Yes (SNL @ Min. Gate)
674.2	41.2	15,400	75	0	Marginal (SNL)
670.0	37.0	19,650	100	30,400	Marginal
660.9	27.9	18,800	100	0	Marginal (SNL)

Table A4. Model Turbine Performance at Maximum Runner Blade Angle (32 degrees) and Tailwater Elevation of 624 feet msl

Pool Elevation (feet msl)	Gross Head (feet)	Flow (cfs)	Wicket Gate Opening (%)	Power (horsepower)	Operational Feasibility
681.0	57.0	21,500	100	99,425	Yes
681.0	57.0	16,800	75	47,850	Yes
681.0	57.0	12,700	57	0	Yes (SNL @ Min. Gate)
670.0	46.0	20,500	100	61,650	Yes
670.0	46.0	15,850	75	15,500	Yes
665.2	41.2	15,400	75	0	Marginal (SNL)
660.0	36.0	19,550	100	26,750	Marginal
652.2	28.2	18,650	100	0	Marginal (SNL)

In addition to the above measured data, qualitative information was obtained through direct observation to identify effects on stability of operation. These observations indicated that a vortex was formed on the runner for various conditions measured above (see Figure A6). The vortex is an indication of undesirable and possibly unsafe zones of turbine operation. The development of a vortex normally corresponds with severe unstable conditions. The vortex forming and collapsing creates pressure pulsations and causes severe vibrations from unstable flow distribution to the runner.

Froude modeling techniques do not allow investigation of cavitation phenomena. However, significant cavitation can be expected to occur, increasing the tendencies for unstable operation at high flows and low tailwater conditions. Severe cavitation could also cause damage to the machinery and structures.

At head ranges far outside the design operating range, the turbines operate at reduced efficiencies. The poor performance of the turbine (low efficiency) indicates the equipment and structure must absorb substantial energy. For example, at the minimum gate SNL condition (zero turbine efficiency) with the minimum blade angle of 20 degrees and a flow of 218 m^3/s (7,700 cfs), 37 megawatts of potential energy must be dissipated through the turbine and powerhouse structure.

The observations indicated that the worst conditions of unstable operation and vortex formation occurred with a blade angle of 32 degrees, wicket gate openings from 100 percent to 75 percent, and heads of 14 m (46 feet) and below. The worst condition noted during the observational testing was for 100 percent wicket gate opening, 32 degrees blade angle, tailwater of 190 m (624 feet) msl, and gross head of 8.6 m (28.2 feet) (SNL condition). As head on the turbine is reduced with a blade angle of 20 degrees (for either tailwater), the model testing indicates acceptable to marginal operating conditions.

The effect of lowering the pool elevation on the turbine intake velocity was noted during the observational testing. As the pool elevation was lowered and intake flow area decreased, the velocity increased. The study team also noted that higher intake velocities may cause higher loading on the trash racks from debris accumulation, which may affect the turbine discharge capacity.

A.2.3 Recommendations for Using Existing Turbines

Turbine Operation

Tables A1 through A4 show which operating conditions are operationally feasible. The turbine discharge capabilities for those operating conditions are used to evaluate various drawdown alternatives. However,

unstable or unacceptable operation may occur at many of the conditions identified in the tables, which may preclude actual operation at those conditions. The magnitude of the response of the actual turbine to the hydraulic conditions is difficult to quantify for zones of turbine operation far below accepted design practice.

Because the actual response to operation far below the design range is uncertain, operation to the SNL condition should be restricted to low blade angles and should be carefully monitored prior to incremental increases in discharge.

Operation below SNL is possible, but would require direct manual operation of each turbine. The turbine generators must be disconnected from the power grid by opening the breakers. Operation below SNL would require an operator at each turbine to adjust the wicket gates and monitor the turbine speed and other unit parameters. More critical evaluation of this option is necessary to establish the operating methods and constraints.

Performance Instrumentation

The turbines and plant should be appropriately instrumented to detect structurally dangerous conditions. Instrumentation should measure acceleration, shaft run out, increased leakage, bearing temperatures, structural and mechanical vertical displacement, and pressures at the head cover, intake, and draft tube man doors. There should also be instrumentation to detect runner blade impact on the discharge ring. The study team recommends installing instrumentation for one turbine unit and conducting several tests to make sure the instrumentation setup is sufficient and working properly before the instrumentation is installed on the remainder of the units. Less instrumentation would be required for bladeless runner units, but some instrumentation would still be necessary.

Emergency Closure Devices

Existing emergency closure devices should be in operating condition. Currently, the intake gates at each project are either raised (with the hydraulic operators disconnected) or removed for improved fish passage purposes. Figure A12 shows a typical powerhouse intake gate. During a reservoir drawdown, the fish screens would be removed. The intake gates should be connected to the hydraulic operators and stored in the normal position, ready for emergency use.

Cooling Water System

Additional cooling water for turbines and generators would be required to supplement the existing gravity-fed system as the head drops. There are two broad categories of water that need to be provided, depending on absolute pool level and whether generation is necessary. The first category is the water required for thrust- and guide-bearing cooling, gland water, air compressors, station service transformers, and heat pumps for cooling the control and computer rooms. This water is required as long as the units are turning, whether they are generating or not. The bearing cooling water can be shut off if the units are stationary. The second category is for cooling water for the main unit. This cooling water is required only if the units are generating. The main unit transformers are air-cooled.

The following modifications would be typical to adapt the existing turbine cooling water system for drawdown conditions:

- Provide a piping header from an external source providing 57 m^3 (15,000 gallons) per minute to supply cooling water pumps.
- Install six generator-cooling water pumps, motors, and pump bases to replace the existing gravity-feed system.

- Install six thrust- and guide-bearing cooling water pumps, motors, and pump bases to replace the existing gravity-feed system.
- Install electric power supply for the pump motors.
- Provide remote annunciation and operation in the powerhouse control room.

Trash Rack Modifications

Investigation is necessary to assure that the trash rack structures are adequate for the wide range of static and dynamic loads anticipated. Some strengthening has been assumed to be necessary for drawdown conditions. The trash racks should be inspected and modifications made as necessary prior to drawdown. A significant effort will be required to keep the trash racks clear of debris during drawdown.

Draft Tube Bulkheads

If more than one project is drawn down at once, the tailwater of the upstream project will drop significantly. For example, normal minimum tailwater at Lower Granite is 193 m (633 feet). If Little Goose Reservoir is also drawn down, the tailwater at Lower Granite will fall to about 190 m (624 feet). This drop in tailwater may cause serious cavitation problems for the turbines. One solution is to minimize cavitation conditions by partially lowering the draft tube bulkheads to create an orifice in the draft tube. This would increase head losses and create an artificial tailwater for the turbines. More specific model studies are necessary to establish the parameters to best prevent potential problems. It is not intended that extraordinary measures be implemented to prevent all damage. The intention is to control the rate of damage progression during the critical periods of operation.

The loading on the bulkheads and supporting structures would be in the opposite direction from how they were designed, and the forces would no longer be just static loading. Figure A13 shows existing draft tube bulkheads. A more complete structural analysis that would include a vibration analysis and a review of hydraulic pull-down forces on the bulkheads would need to be completed before implementing this action. Each project has only one set of draft tube bulkheads, so additional bulkheads for the remaining five units would need to be fabricated.

A.3 Waterways Experiment Station Model Testing

WES performed hydraulic turbine model testing using a 1:25 scale sectional model of the Lower Granite Dam turbines with the blade removed. Numerous experiments were performed at a tailwater of 190 m (624 feet) msl to determine the capacity of the turbine unit at various forebay pool elevations and to give some indication of the bladeless turbine operating conditions. Both velocity and pressure data were collected. Curves were developed to help determine how to operate the unit to achieve the desired drawdown rates. These curves are straightforward and were repeatable.

A total of 10 velocity experiments were performed to document what flow conditions might exist at various operation points that might occur during the drawdown process. Velocities were measured in the intake structure upstream of the wicket gates in Bays A and B, in the vicinity of the bladeless runner, and in both barrels of the draft tube.

A.3.1 Results

Velocities Upstream of the Turbine

See Figure A7 for typical velocity measurement locations upstream of the turbine. The velocity experiments indicated that no problems would be anticipated for flow conditions upstream of the wicket gates. This is true for all heads and discharges, as long as the trash racks and wicket gates are free of

debris. As the reservoir is drawn down, debris loading may increase. Velocities and flow conditions in the turbine intakes will be affected if debris accumulates on the trash racks. Care must be taken to keep the trash racks clean for the information presented here to be valid.

Velocities measured 1.2 m (4 feet) upstream of the upstream edge of the wicket varied between 4.0 m (13.2 feet) per second for a turbine discharge of 600 m^3/s (20,000 cfs) to 1.5 m (5 feet) per second for a turbine discharge of 200 m^3/s (7,000 cfs). Since the flow is accelerating downstream, it can be expected to reach a velocity of 15.5 m (50.7 feet) per second for a turbine discharge of 600 m^3/s (20,000 cfs) to (14.8 feet) per second for a turbine discharge of 200 m^3/s (7,000 cfs) with an upper pool of 195 m (640 feet) at the controlling point of the wicket gate opening. Although high velocities would be expected in this area with a bladed runner, the velocities between the wicket gates are even higher than what would be expected. The measured velocities upstream of the wicket gates indicated no instabilities in the approaching flow field.

Velocities Near the Bladeless Runner

See Figure A8 for typical velocity measurement locations near the bladeless runner. Velocities measured in the vicinity of the runner indicated that high velocities could be expected at this location for discharges of 600 m^3/s (20,000 cfs) and 400 m^3/s (15,000 cfs). Measured velocities for a turbine loading of 600 m^3/s (20,000 cfs) were as high as 28.5 m (93.4 feet) per second near the runner. This was much higher than expected. At this measurement section, the average velocity based on the available area would be approximately 14 m (46 feet) per second. Measured velocities for lower discharges were as high as 17 m (57 feet) per second. This is also a high velocity but is close to what would be expected at some of the normal turbine operations with a bladed runner.

Unstable flow conditions are likely to occur when there are extremely high velocities due to cavitation and severe turbulence. Large vibrations occurred in the model for discharges of 600 m^3/s (20,000 cfs). While these vibrations are not scaleable, they indicate operating conditions that would not be good for the unit, even for short durations.

Velocities in the Draft Tube

See Figures A9 and A10 for typical velocity measurement locations in the draft tube. Velocities measured in the draft tube indicated a very non-uniform flow field. This was true for all 10 experimental conditions. These conditions should not prohibit the use of the bladeless runner for the natural river drawdown process. However, for all 10 conditions, the boil that occurs downstream in the tailrace area occurred much further downstream than was expected. The boil in the Voest-Alpine MCE bladed runner model occurred much closer to the structure than it did in the WES model. The bladed runner removes much of the energy in the flow to produce power with the generator. With a bladeless runner, most of this energy remains in the flow, so the boil occurs much further downstream. This may have an effect on scour in the tailrace area downstream of the draft tube exits, depending on how well the bed is armored.

Pressure Experiments

Pressures were measured at five locations in the vicinity of the bladeless runner as shown in Figure A11. Four of these locations were on the discharge ring, and one was located on the bladeless runner itself. These experiments indicated that cavitation will occur on the bladeless runner for discharges of 600 m^3/s (20,000 cfs). The tests also indicated that there is potential for cavitation on the runner for discharges of 440 m^3/s (15,600 cfs) and 400 m^3/s (15,000 cfs). These pressure readings are consistent with the extremely high velocity measurements noted for these discharges. The cavitation associated with the 400 m^3/s (15,000 cfs) range is probably not of the magnitude to prohibit operating the bladeless unit for the length of time it would take to draft the reservoir down to elevation 195 m (640 feet) msl.

A.3.2 Recommendations for Using Bladeless Turbines

The bladeless runner can be used to draw down the river to the lowest possible river levels. Approximately 4 m to 7 m (12 to 22 feet) of head is required to yield the required discharge through the turbine units. Curves generated from the hydraulic model studies show that the potential hydraulic capacity of a unit with a bladeless runner is higher than that of a unit with a normal bladed runner. However, at discharges above approximately 400 m^3/s (15,000 cfs), there would be problems with extremely high velocities, cavitation, vibrations, and unstable flow conditions. Potential cavitation problems may be encountered at discharges in the range of 400 m^3/s (15,000 cfs). These problems are most pronounced at high heads. It is recommended that the maximum discharges of the unit not exceed 400 m^3/s (15,000 cfs). If possible, the maximum unit discharge should be closer to 300 m^3/s (10,000 cfs). Limiting the discharge may also reduce the likelihood of scour problems in the tailrace area downstream of the draft tube exits.

With the turbine and generator units in place, there is not enough clearance to remove the blades intact. The blades must be cut off at the hub and removed through the draft tubes. The cut surface should be made smooth with the surface of the hub to minimize cavitation.

The high velocities and associated turbulence through the bladeless runner area would not be conducive to safe fish passage. The bladeless runner should not be used if migrating juvenile fish are in the river.

The recommendations noted earlier for the existing turbines regarding emergency closure systems and trash rack cleaning are also applicable for the bladeless runner units.

A.4 Proposed Configuration

The maximum drawdown rate has been established at 0.6 m (2 feet) per day. To achieve this drawdown rate, the turbines must pass a flow amount equal to the discharge necessary to draft the reservoir plus the inflow into the reservoir. Figure 2 shows summary hydrographs for Ice Harbor Dam, which is representative of all four lower Snake River dams for the purposes of this report. The graph shows that the average mean daily flow for the period from August through November is 850 m^3/s (30,000 cfs) or less. The maximum mean daily discharge for most of this period (up through about mid-November) is 1,400 m^3/s (50,000 cfs) or less.

Tables A5 and A6 show the required discharge for various heads at Lower Granite Dam as the reservoir is drafted for inflows of 850 m^3/s (30,000 cfs) and 1,400 m^3/s (50,000 cfs). The tables also show the hydraulic capacity of an existing turbine and a bladeless runner unit for each head, along with a possible combination of the two types of units to satisfy the required total discharge.

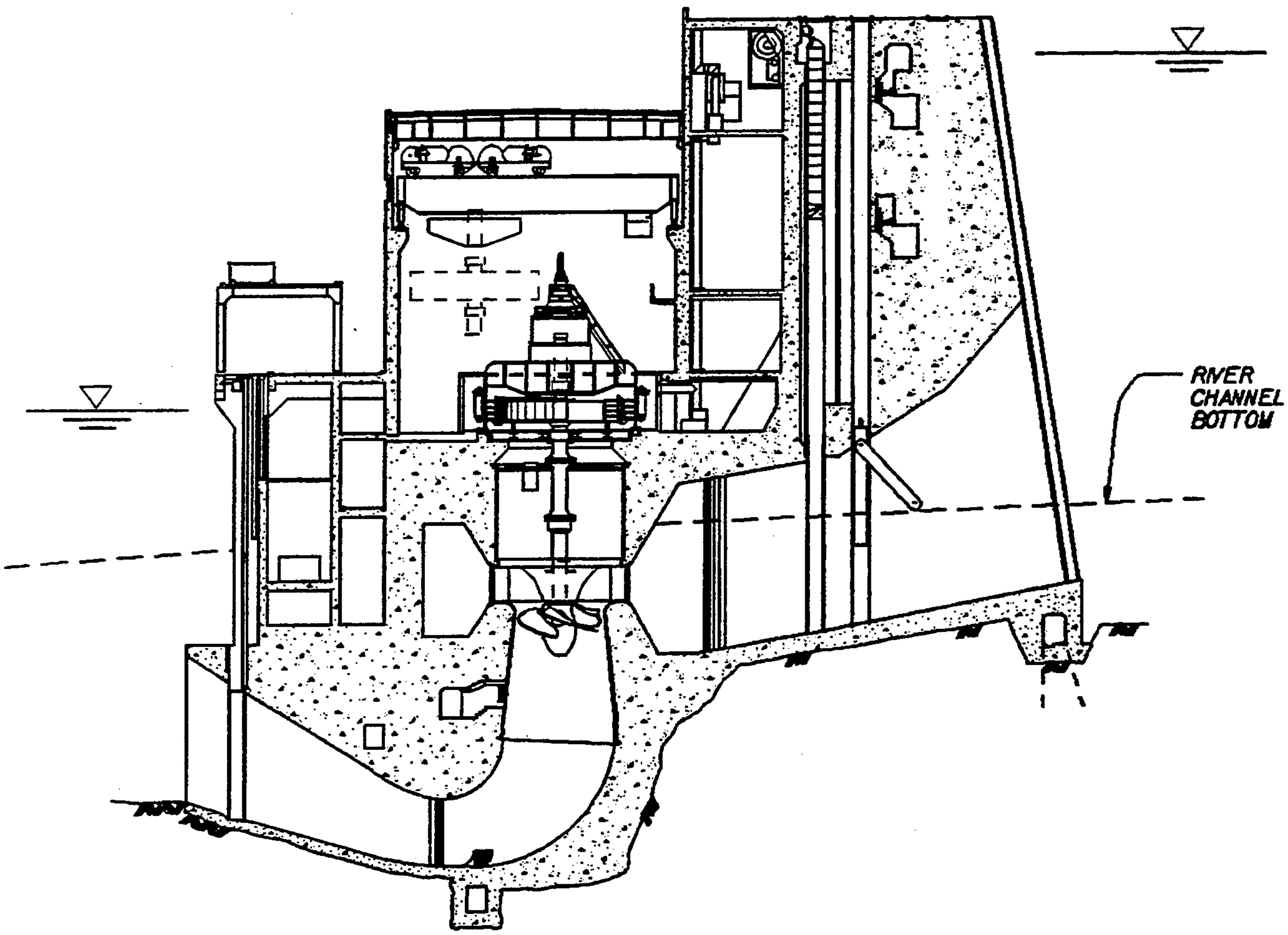
The best combination appears to be three existing turbines and three units with bladeless runners. At the high heads, the entire discharge can pass through the three existing turbines. It is best of avoid using the bladeless runner units at high heads, even with restricted discharge. The existing turbines reach the SNL condition with the wicket gates 100 percent open at a head of about 6.2 m (20.2 feet). It is possible to operate the turbines below the SNL level, as described in Section A.2.3 for the existing turbines, allowing the reservoir to be drafted lower than 6 m (20 feet) of head. However, operation below the SNL is not recommended unless necessary. In most years, depending on river inflow, the three bladeless runner units would be sufficient to draft the reservoir below the 6-m (20-foot) level, and possibly as low as 3 m (10 feet), without exceeding the 280 m^3/s (10,000 cfs) limit per unit. More bladeless runner units would ensure the ability to draft the reservoir to the very low heads, but would also make it necessary to start using the bladeless runner units at higher heads while reducing the benefits derived by using the existing turbines. Using fewer bladeless runner units would reduce the ability to lower the final drawdown head below the SNL level (about 3 m [20 feet]).

Table A5. Required Discharges and Proposed Turbine Configurations for Lower Granite with Inflow of 30,000 cfs and Tailwater Elevation of 624 feet

Forebay Elevation (feet msl)	Head (feet)	Reservoir Volume (AF)	Drawdown Discharge (cfs)	Total Required Discharge (cfs)	Maximum Discharge for Existing Turbine (cfs)	Minimum Discharge for Existing Turbine (cfs)	Target Maximum Discharge for Bladeless Runner Turbine (cfs)	Proposed Operating Configuration
733	109	442,900	8,067	38,067			10,000	2EX, 0BL
730	106	418,900	8,067	38,067	19,400	15,700	10,000	2EX, 0BL
725	101	380,900	7,663	37,663	19,686	15,744	10,000	2EX, 0BL
720	96	345,200	7,200	37,200	20,231	15,460	10,000	2EX, 0BL
715	91	312,000	6,695	36,695	20,279	15,370	10,000	2EX, 0BL
710	86	281,000	6,252	36,252	19,968	15,410	10,000	2EX, 0BL
705	81	252,100	5,828	35,828	18,875	13,350	10,000	2EX, 0BL
700	76	225,100	5,445	35,445	18,350	13,475	10,000	2EX, 0BL
695	71	200,200	5,022	35,022	17,900	13,700	10,000	2EX, 0BL
690	66	177,100	4,659	34,659	17,400	13,750	10,000	2EX, 0BL
685	61	155,800	4,296	34,296	17,200	13,850	10,000	2EX, 0BL
681	57	140,200	3,933	33,933	15,000	7,700	10,000	3EX, 0BL
680	56	136,500	3,731	33,731	14,950	7,800	10,000	3EX, 0BL
675	51	119,000	3,529	33,529	14,700	8,150	10,000	3EX, 0BL
670	46	103,400	3,146	33,146	14,450	8,550	10,000	3EX, 0BL
665	41	89,700	2,763	32,763	14,150	9,000	10,000	3EX, 0BL
660	36	77,800	2,400	32,400	13,850	9,550	10,000	3EX, 0BL
655	31	67,900	1,997	31,997	13,500	10,300	10,000	3EX, 0BL
650	26	59,800	1,634	31,634	13,200	11,100	10,000	2EX, 1BL
645	21	53,200	1,331	31,331	12,900	12,700	10,000	2EX, 1BL
640	16	46,600	1,331	31,331	NA	NA	10,000	0EX, 3BL
635	11	42,900	746	30,746	NA	NA	10,000	0EX, 3BL

Table A6. Required Discharges and Proposed Turbine Configurations for Lower Granite with Inflow of 50,000 cfs and Tailwater Elevation of 624 feet

Forebay Elevation (feet msl)	Head (feet)	Reservoir Volume (AF)	Drawdown Discharge (cfs)	Total Required Discharge (cfs)	Maximum Discharge for Existing Turbine (cfs)	Minimum Discharge for Existing Turbine (cfs)	Target Maximum Discharge for Bladeless Runner Turbine (cfs)	Proposed Operating Configuration
733	109	444,100	8,067	58,067			10,000	3EX, 0BL
730	106	420,200	8,033	58,033	19,400	15,700	10,000	3EX, 0BL
725	101	382,300	7,643	57,643	19,686	15,744	10,000	3EX, 0BL
720	96	346,900	7,139	57,139	20,231	15,460	10,000	3EX, 0BL
715	91	313,900	6,655	56,655	20,279	15,370	10,000	3EX, 0BL
710	86	283,100	6,211	56,211	19,968	15,410	10,000	3EX, 0BL
705	81	254,500	5,768	55,768	18,875	13,350	10,000	3EX, 0BL
700	76	228,000	5,344	55,344	18,350	13,475	10,000	3EX, 0BL
695	71	203,500	4,941	54,941	17,900	13,700	10,000	3EX, 0BL
690	66	181,000	4,538	54,538	17,400	13,750	10,000	3EX, 0BL
685	61	160,300	4,175	54,175	17,200	13,850	10,000	3EX, 0BL
681	57	145,100	3,832	53,832	15,000	7,700	10,000	3EX, 1BL
680	56	141,500	3,630	53,630	14,950	7,800	10,000	3EX, 1BL
675	51	124,500	3,428	53,428	14,700	8,150	10,000	3EX, 1BL
670	46	109,400	3,045	53,045	14,450	8,550	10,000	3EX, 1BL
665	41	96,100	2,682	52,682	14,150	9,000	10,000	3EX, 1BL
660	36	84,500	2,339	52,339	13,850	9,550	10,000	3EX, 2BL
655	31	74,700	1,976	51,976	13,500	10,300	10,000	3EX, 2BL
650	26	66,700	1,613	51,613	13,200	11,100	10,000	3EX, 2BL
645	21	60,200	1,311	51,311	12,900	12,700	10,000	3EX, 2BL
640	16	53,600	1,331	51,331	NA	NA	10,000	
635	11	49,900	746	50,746	NA	NA	10,000	



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER GRANITE POWERHOUSE SECTION

Figure: A1

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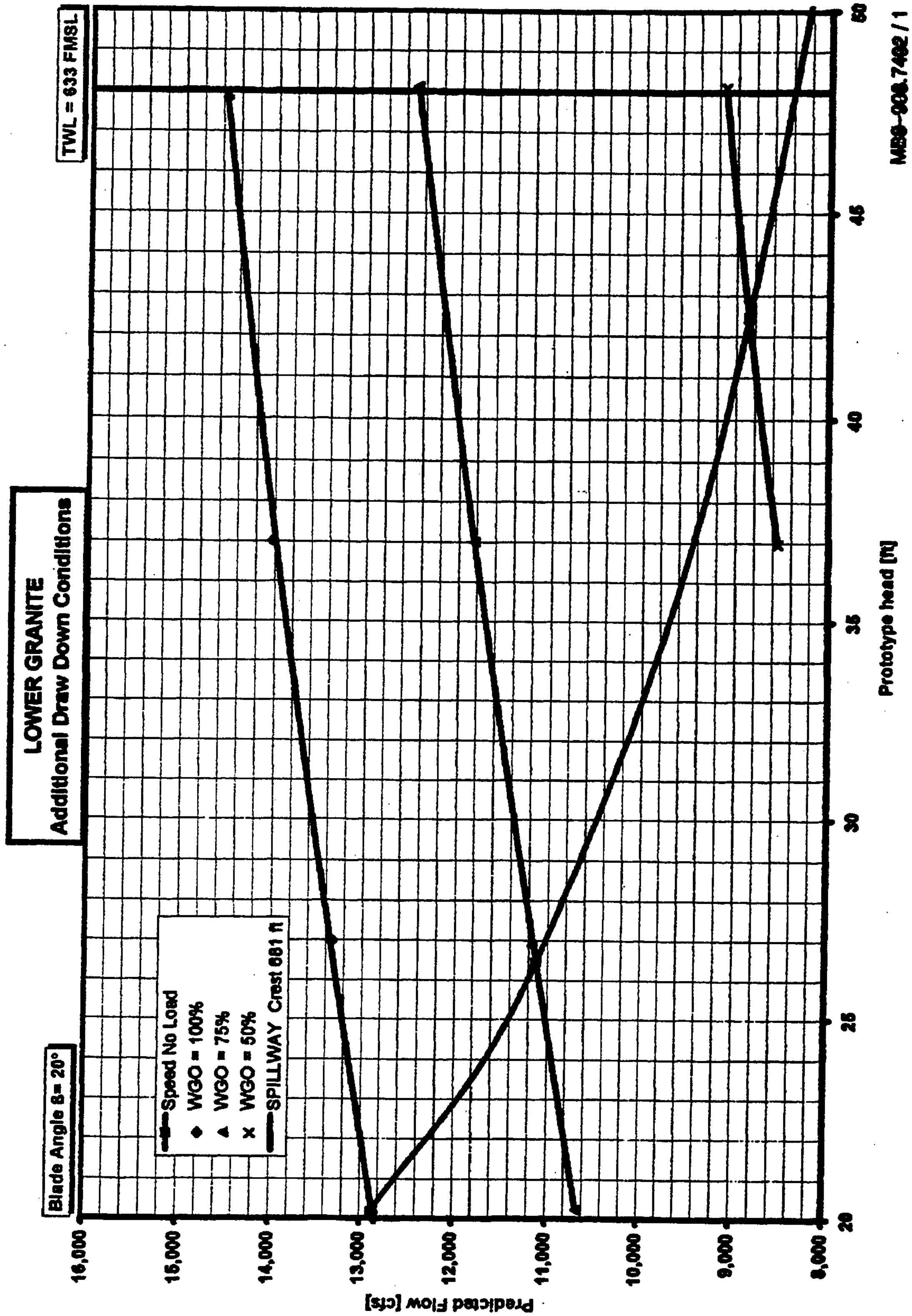
SHEET MAIN SCALE:



US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TURBINE PERFORMANCE CURVES FOR LOWER GRANITE
BLADE ANGLE=20, TW=633'

Figure:
A2

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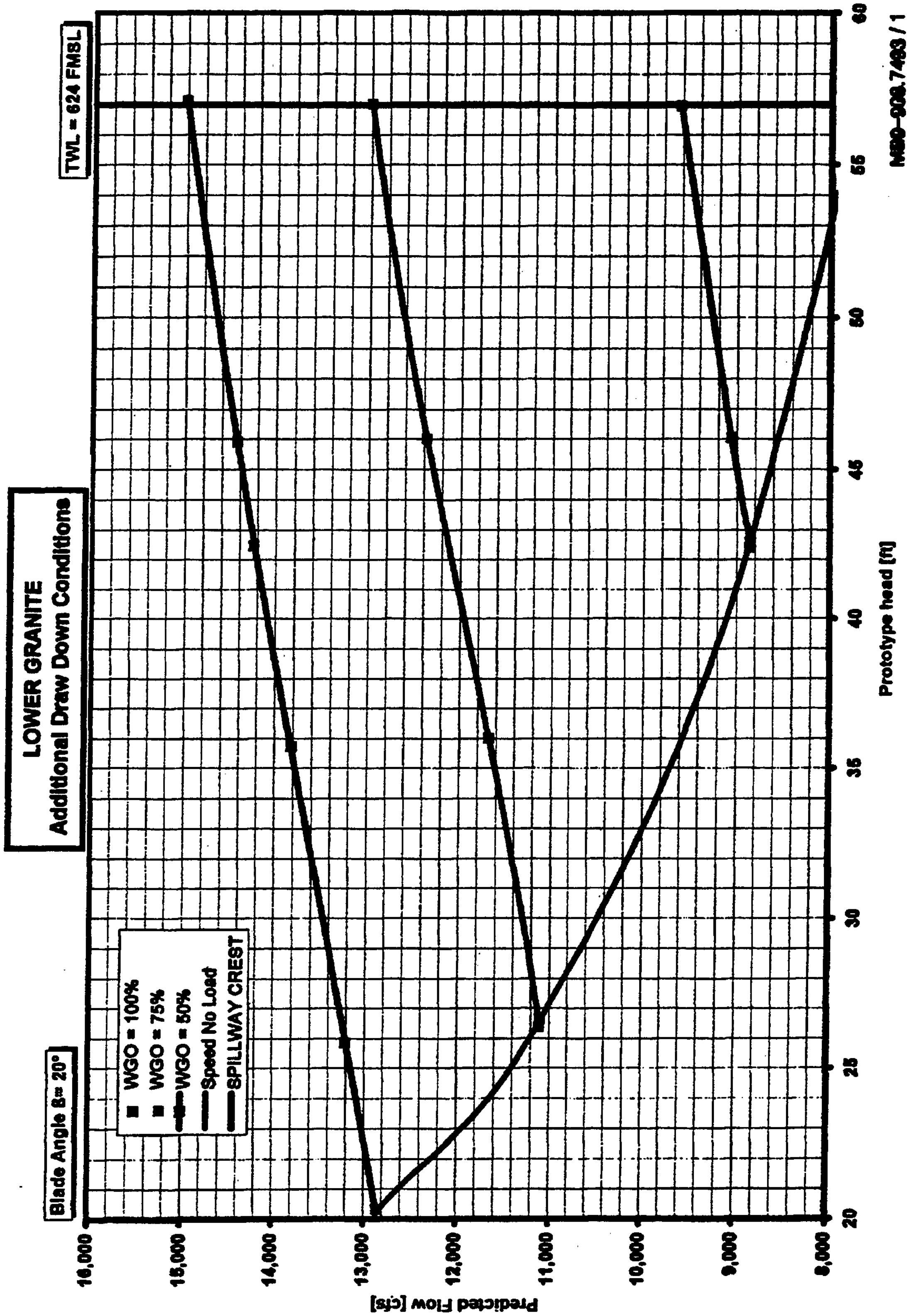

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Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TURBINE PERFORMANCE CURVES FOR LOWER GRANITE
BLADE ANGLE=20, TW=624'

Figure:
A3

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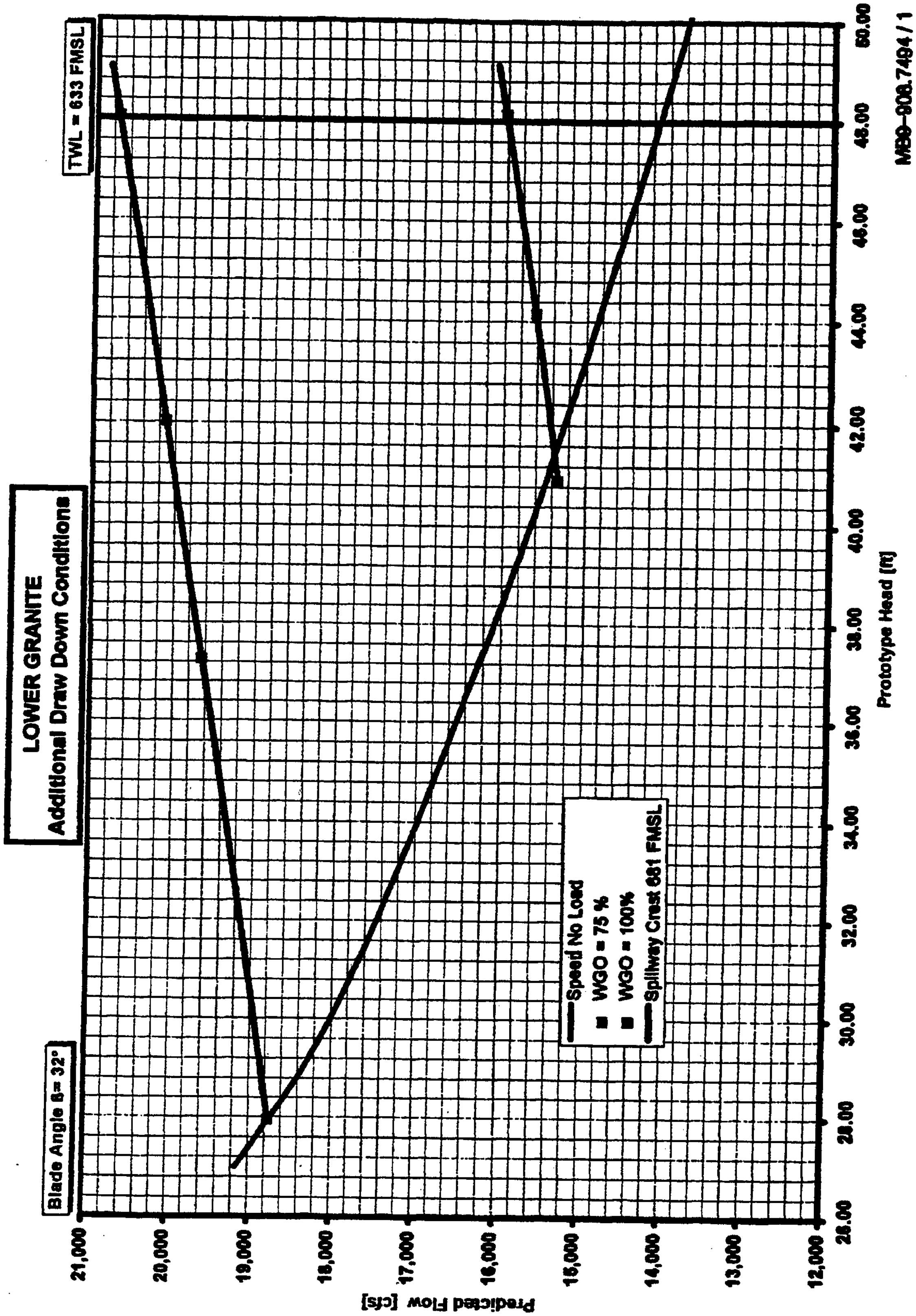

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Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TURBINE PERFORMANCE CURVES FOR LOWER GRANITE
BLADE ANGLE=32, TW=633'

Figure:
A4

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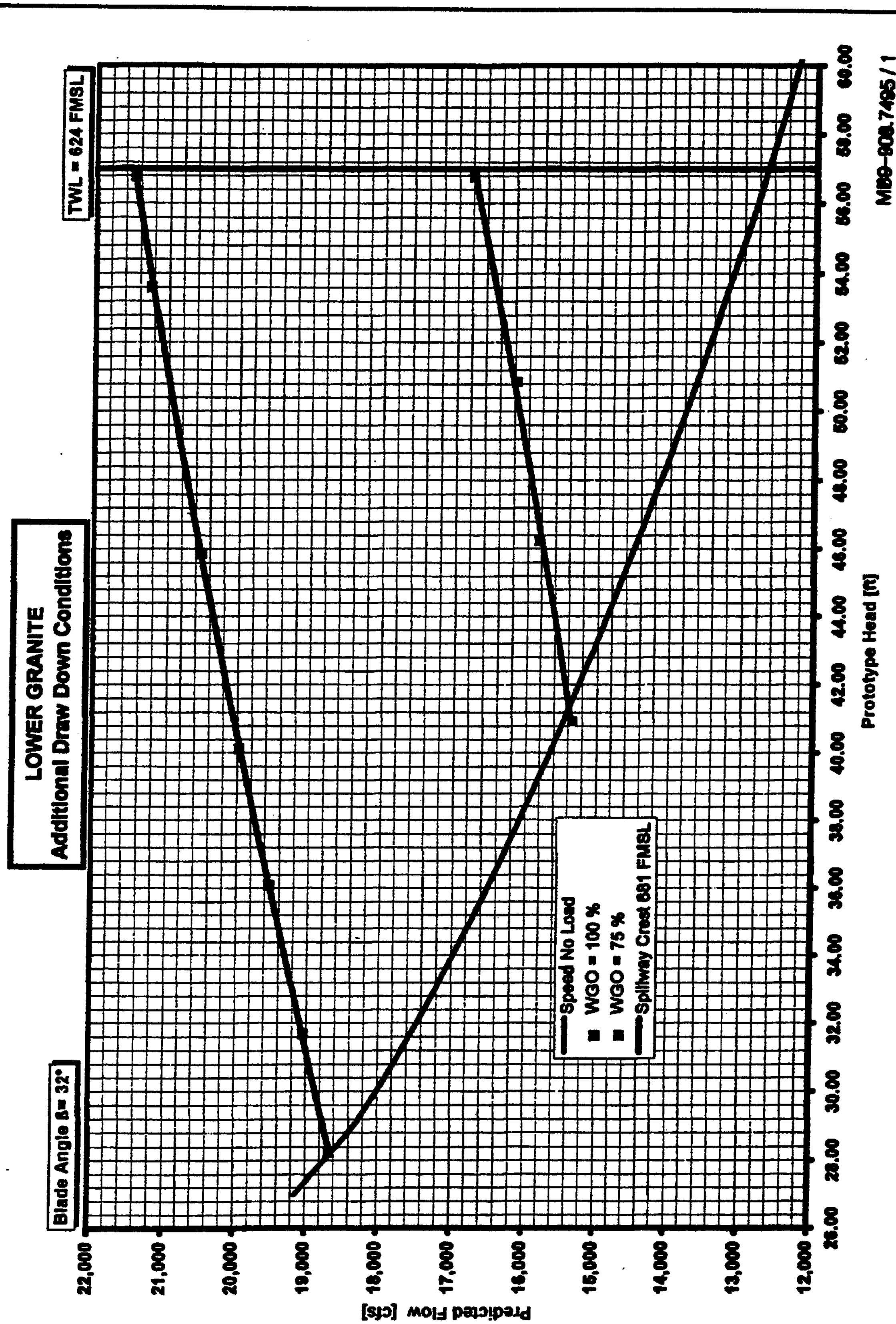

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TURBINE PERFORMANCE CURVES FOR LOWER GRANITE
BLADE ANGLE=32, TW=624'

Figure: A5

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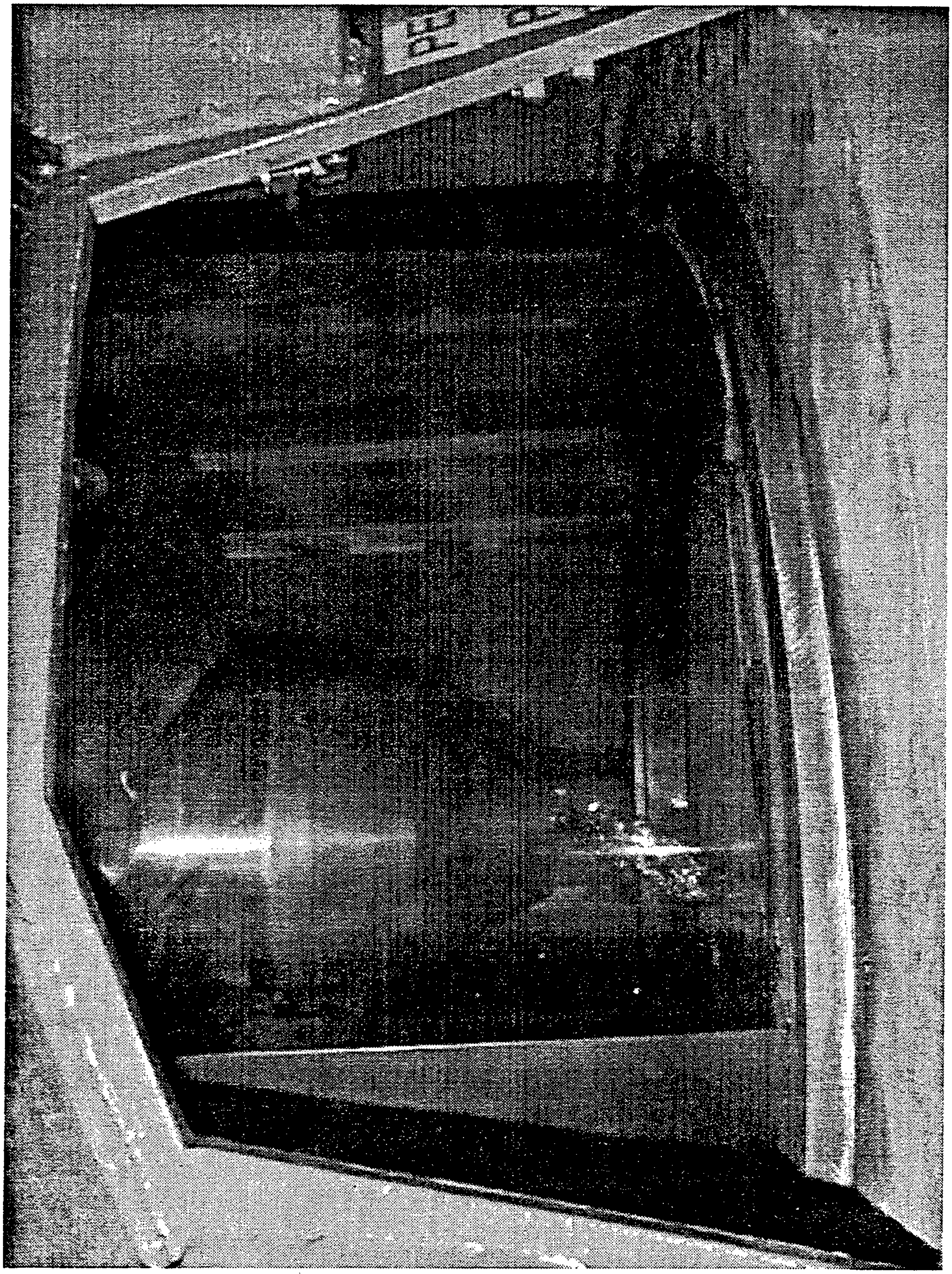

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
VORTEX IN THE LOWER GRANITE TURBINE MODEL
AT VOEST ALPINE MCE

Figure:
A6

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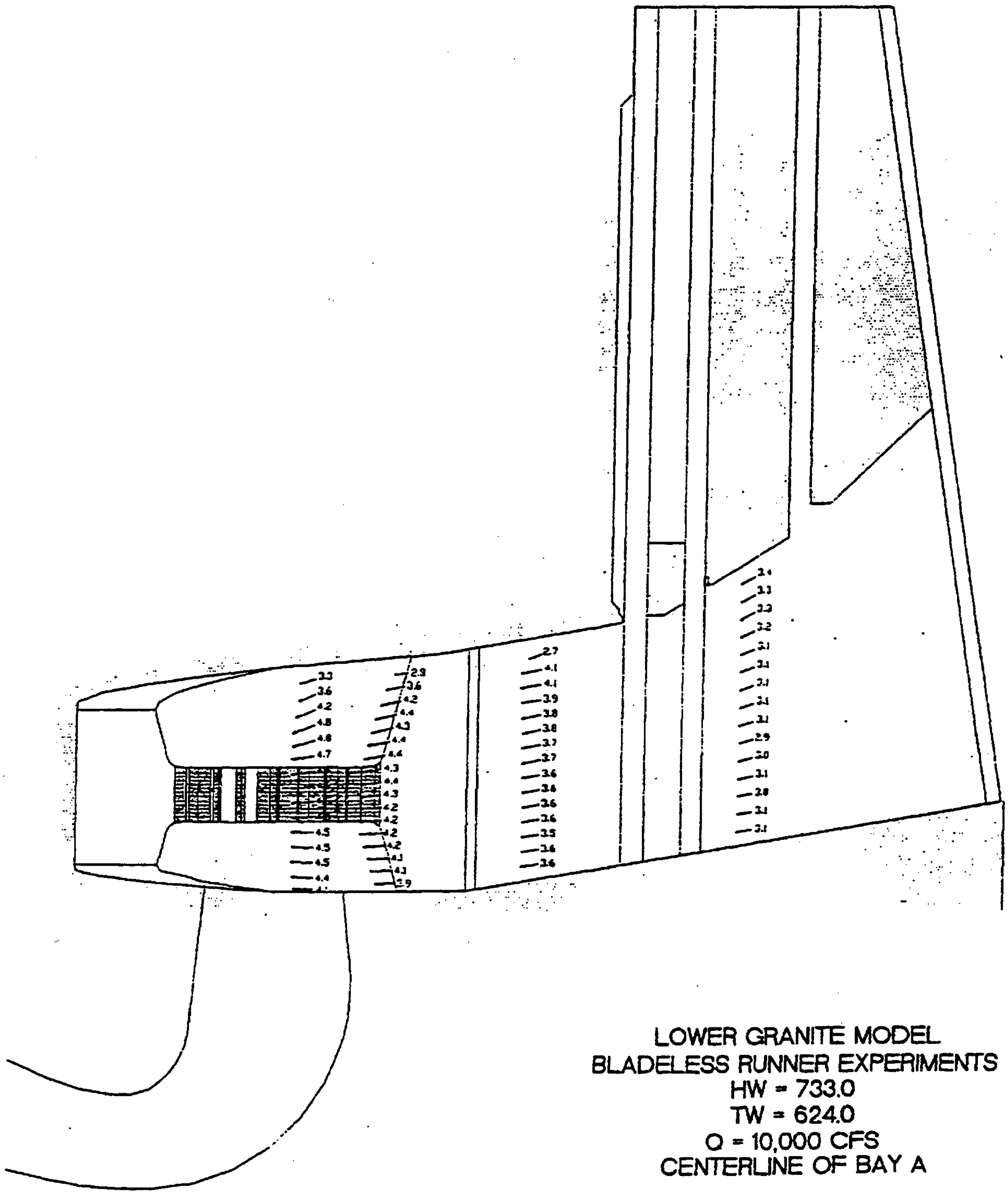

LOWER GRANITE MODEL
BLADELESS RUNNER EXPERIMENTS
HW = 733.0
TW = 624.0
Q = 10,000 CFS
CENTERLINE OF BAY A

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TURBINE INTAKE VELOCITIES WITH A BLADELESS
RUNNER AT LOWER GRANITE

Figure:
A7

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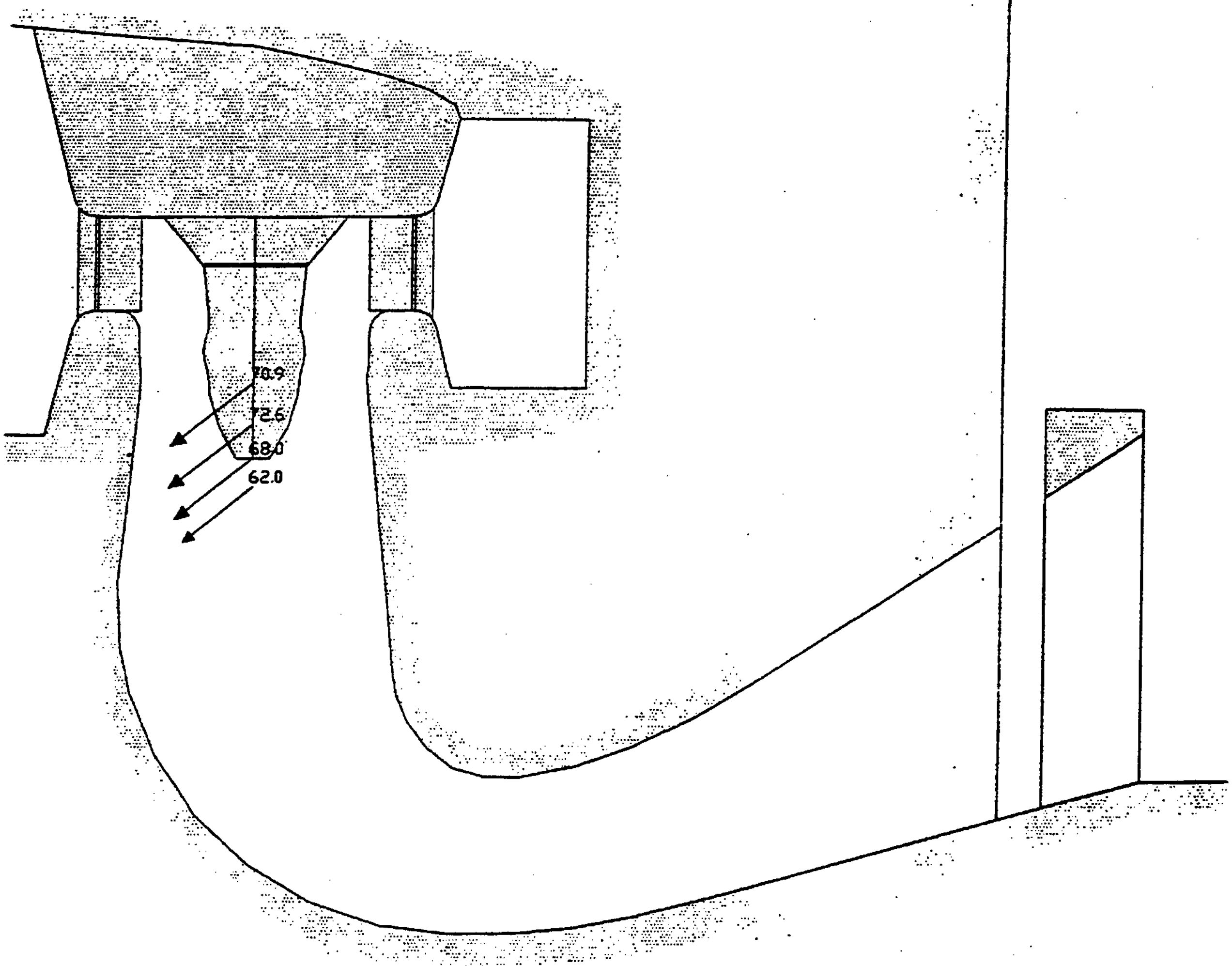

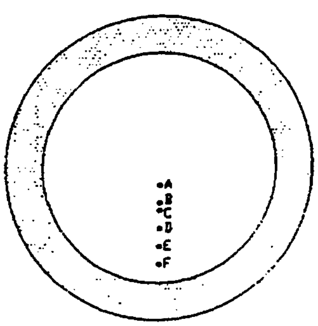

LOCATION OF VEL MEASUREMENTS
RUNNER NOT SHOWN

LOWER GRANITE MODEL
BLADELESS RUNNER EXPERIMENTS
HW = 733.0
TW = 624.0
Q = 10,000 CFS
LOCATION F

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

VELOCITIES IN THE VICINITY OF THE BLADELESS RUNNER AT LOWER GRANITE

Figure: A8

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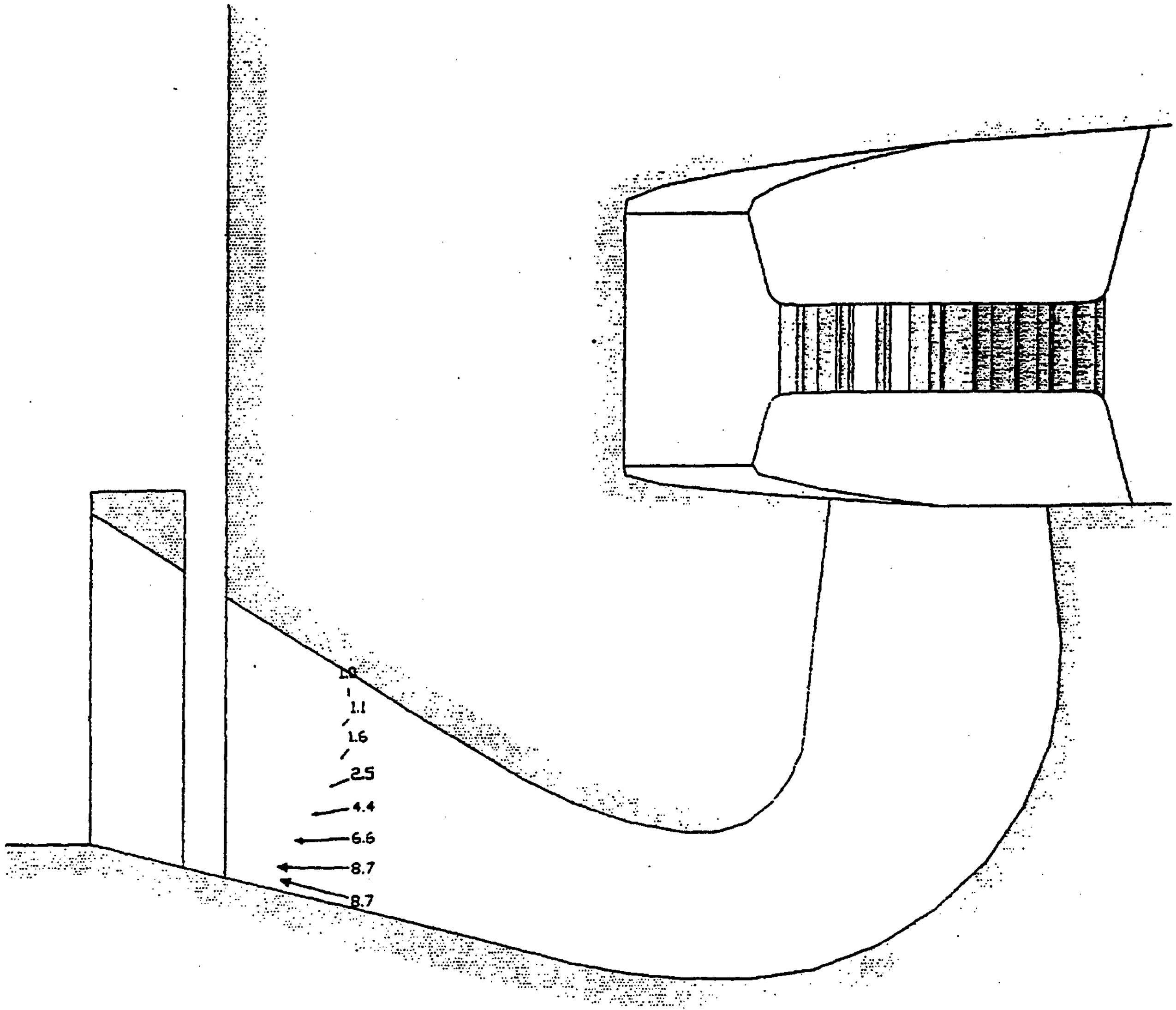

LOWER GRANITE MODEL
BLADELESS RUNNER EXPERIMENTS
HW = 733.0
TW = 624.0
Q = 10,000 CFS
Z = 17.75 FT
BARREL A

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

VELOCITIES IN BARREL A OF THE DRAFT TUBE
AT LOWER GRANITE

Figure:
A9

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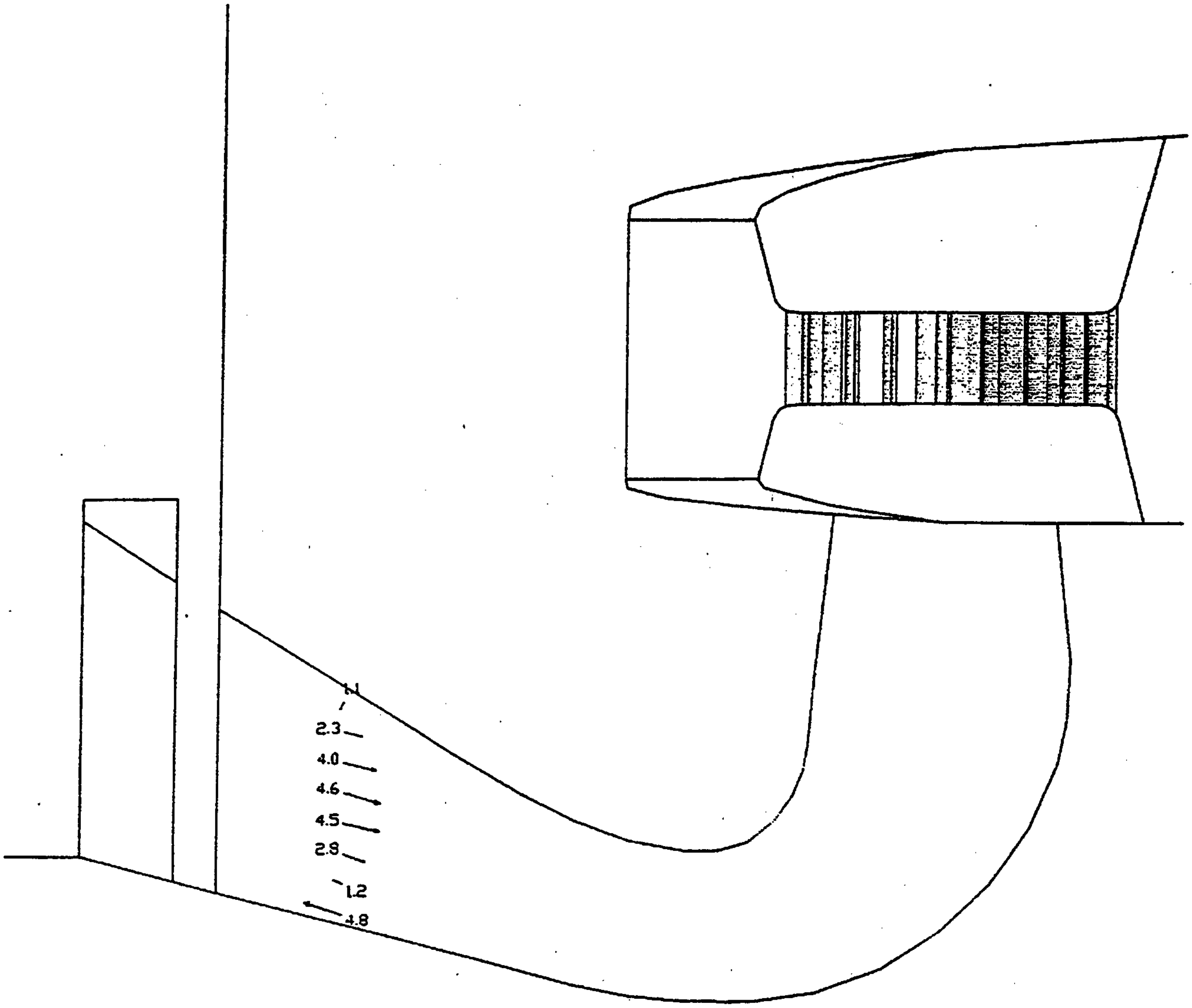

LOWER GRANITE MODEL
BLADELESS RUNNER EXPERIMENTS

HW = 733.0
TW = 624.0
Q = 10,000 CFS
Z = 59.25 FT
BARREL B

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

VELOCITIES IN BARREL B OF THE DRAFT TUBE
AT LOWER GRANITE

Figure:
A10

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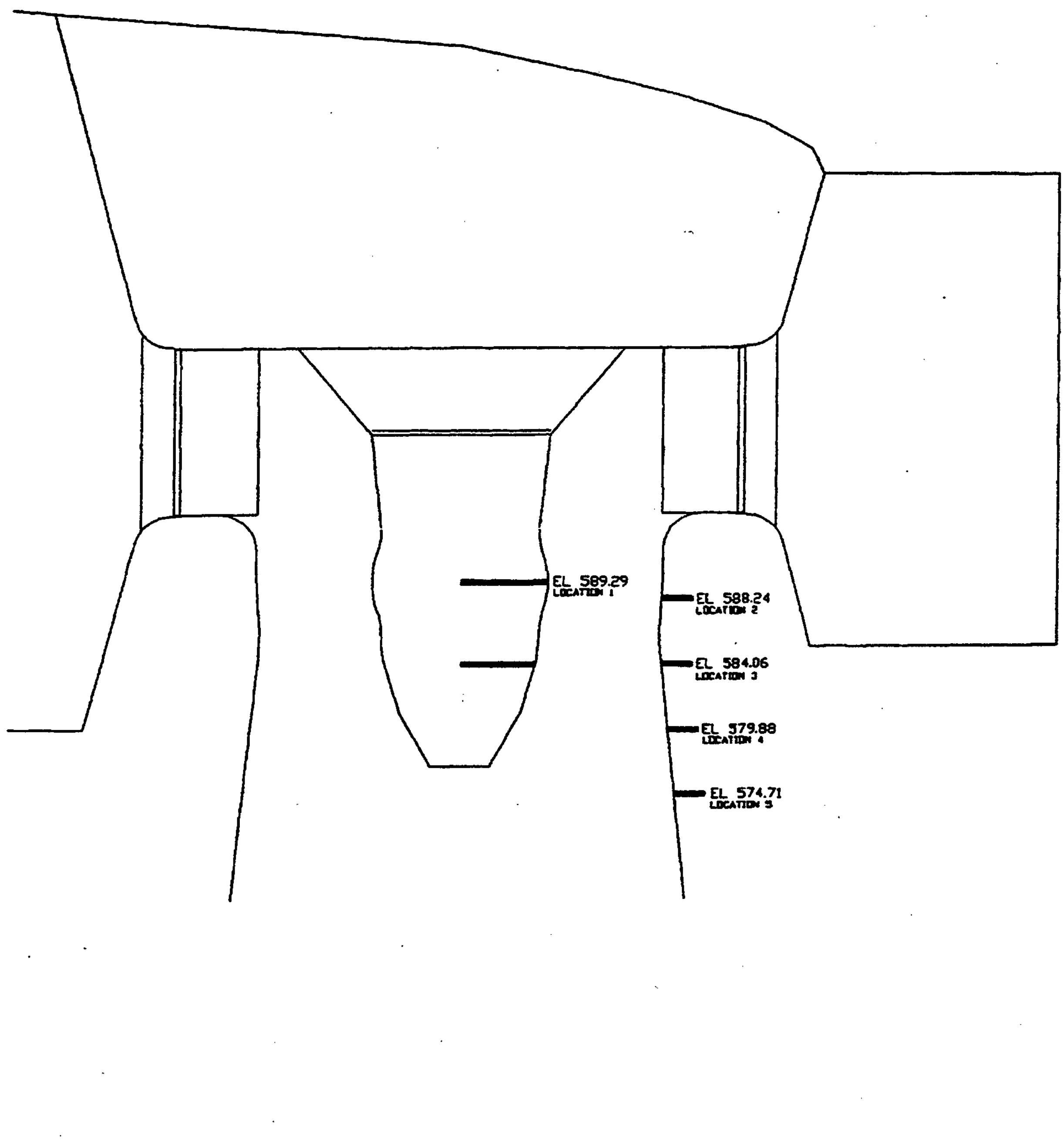

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER GRANITE MODEL BLADELESS RUNNER EXPERIMENTS
PRESSURE MEASUREMENT LOCATIONS

Figure: A11

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Annex B

Dam Embankment Excavation Plan

Annex B: Dam Embankment Excavation Plan

B.1 General

This annex describes the new channels that would be excavated around each of the four lower Snake River dams to provide free-flowing river conditions. The concepts for this dam excavation plan derive from a separate report prepared for the Corps by Raytheon Infrastructure, Inc., titled *Embankment Excavation, River Channelization, and Removal of Concrete Structures* (Raytheon 1998). A general plan of the proposed new channel arrangements for each site is shown on Figures B1 through B4.

The new channels are formed by excavation of the dam embankments or the abutments and construction of levees to channelize the river. The extent of excavation is determined such that average velocities at the sides of the new channel would be well below the upper limit of acceptable fish migration velocities. Fish passage features, however, would be needed to provide resting places for fish during upstream migration because, at times, velocities are above the lower velocity limit against which fish can swim for long distances. The navigation locks and the spillways would not be used to supplement the hydraulic capacity of the new channels for two reasons: 1) they could not be modified within the eight-month construction window, and 2) modifications to these structures would be more costly than adding fish passage features to the new channels.

The total volume of excavation required for the new channels, comprised of embankment excavation and common excavation from the abutments, is shown in Table B1.

Table B1. Total Excavation Quantities for the New Channels (Embankment and Common)

	Quantity (m^3)				
Material	**Lower Granite**	**Little Goose**	**Lower Monumental**	**Ice Harbor**	**Total**
Embankments Only					
Core Material	240,200	138,300	78,300	7,500	464,300
Gravel Fill (shell), Including Rockfill and Riprap	1,101,700	978,000	675,200	59,500	2,814,400
Cofferdams (part of dam)	276,400	263,900			540,300
Embankment Subtotal	1,618,300	1,380,200	753,500	67,000	3,819,000
Common Excavation					
Abutments			4,395,000	3,971,000	8,366,000
Cofferdams (temporary)			172,340	228,480	400,820
Common Subtotal			4,567,340	4,199,480	8,766,820
Total Volumes	1,618,300	1,380,200	5,320,840	4,266,480	12,585,820

B.2 Conditions

All channel inverts under the footprint of the embankment dam would be excavated to form a uniform continuation of the natural river gradient from upstream to downstream. The plan avoids low areas or depressions in the channel floors because a depression would likely be filled in by deposition and, therefore, would change the channel sections and velocities.

At Ice Harbor, Lower Monumental, and Lower Granite dams, the new channels would be located adjacent to and confined by the navigation lock walls on one side, with hillside bordering the other. At Little Goose Dam, the channel would be bounded on one side by the remaining concrete dam structures, with the other side bounded by the hillside.

At Little Goose and Ice Harbor dams, an overbank berm of large riprap would be placed along the river side of both the upstream and downstream levees about halfway to the shore. The berms would be approximately 3 m (10 feet) high and 6 m (20 feet) wide. They would provide scour protection for the toe of the levee. A similar berm would be placed along the base of the navigation lock wall to protect the base of this wall from scour.

Except at Little Goose Dam, guide walls upstream and downstream of the navigation locks would be removed in order to eliminate flow obstructions into and out of the new channel. Guide walls upstream would be floating, and downstream would be concrete anchored into rock. A short portion of the downstream walls would be left in place, with the levee built around it. At Little Goose Dam, this wall would be a tie-in wall for the embankment dam.

At all four sites the new channel's width would be bordered on the land side by the railroads. The top of the channel slope would be offset 12 m (13 yds) toward the river from the centerline of the track. The excavated cut slopes of the new channels that are exposed to flows would be 2.5 horizontal:1 vertical, similar to excavated slopes of the existing channels, and would be 1.5 horizontal:1 vertical above the maximum water surface.

B.2.1 Lower Granite Dam

Removing the north shore embankment would form a free-flowing river channel at Lower Granite Dam. Flow would be diverted from its main course along the south shore to a new channel on the north side of the concrete structures through the area where the embankment is located. This same configuration was successfully used for diversion during the original project construction and, therefore, is expected to work satisfactorily.

At Lower Granite Dam, the natural river channel, which existed before structures were constructed, contained a large island that split the river into two flow channels. The proposed new channel would have a flow area approximately 65 percent of the original river's area; however, the majority of flow previously went through the south channel. Flow would now be diverted to the north of the structures. Since this is hydraulically inefficient, a diversion levee system would be constructed to guide flow smoothly into the new channel and around existing structures.

B.2.2 Little Goose Dam

The new channel at Little Goose Dam would be similar to, but narrower than, the channel at Lower Granite Dam. Removing the north abutment embankment dam would form the channel. At Little Goose, not only is the embankment dam shorter in length than at Lower Granite, but there also is a concrete tie-in wall extending 30 m (98 feet) north of the spillway along the dam's axis. The tie-in wall would be left in place because acceptable velocities would be achieved without removing it. The levee would be arranged to tie into this wall, and the levee's north slope would extend into the new channel. Fish passage features would be required at Little Goose because average flow velocities in the overbank area are higher than 1.5 mps (5 fps).

The main flow course prior to dam construction was on the north side where the new channel would be located.

B.2.3 Lower Monumental Dam

The new channel at Lower Monumental Dam would be on the south side of the river because it would require less excavation than a similar channel on the north side. Flow would be diverted from its original main course along the north shore prior to dam construction to the new channel on the south side of the concrete structures. There is a south embankment dam, but the engineered fill portion of it is only approximately 15 m high (versus 40 m for the total excavation). Therefore, embankment excavation would be only a small portion of the total excavation required for the new channel. The majority of excavation for the new channel would be from the natural soil of the abutment.

The Union Pacific Railroad line exists on the south shore. In order to configure the channel with sufficient width, the existing rail lines must be relocated to the south. The reach of double track has been identified and costs for significant excavation and relocation of rail beds and lines have been developed. It is assumed that funding would be appropriated for such modifications.

The study team planned to leave a temporary cofferdam of natural material along the shoreline to retain the river, thus enabling a significant amount of the excavation to be accomplished in the dry. The natural material of the abutment used for these cofferdams is predominantly sands and gravels. This material could be pervious and significant pumping may be required. An impervious layer of soil may be required on the water side for the cofferdams to function adequately. After excavation of the abutment material, temporary cofferdams along with any required river bed material would be removed by dragline.

B.2.4 Ice Harbor Dam

The new channel at Ice Harbor Dam would be excavated out of the north abutment because the top of the rock on this side is fairly horizontal and excavation would be entirely in soil.

An abandoned rail line, previously owned by Burlington-Santa Fe Railroad exists on the north shore. The railroad right-of-way is currently being developed under the Rails to Trails program. The railroad maintains the right to re-establish the rail line should conditions change. In order to configure the channel with sufficient width, the existing rail lines must be relocated to the north. The reach track has been identified and costs for significant excavation and relocation of rail beds and lines have been developed. It is assumed that the railroad would exercise the option to re-establish the line and that funding would be appropriated for such modifications.

Flow would be diverted from its original main course along the south shore prior to dam construction to the new channel on the north side of the concrete structures.

The location of the new channel would require relocation of the railroad to allow sufficient channel width for acceptable velocities.

To minimize the quantity of "in wet" excavation, temporary cofferdams would remain in place at Ice Harbor, parallel to the shoreline on the river side of the excavation. Material between the cofferdam and the new channel bank would be excavated "in the dry."

B.3 Reservoir Drawdown Issues

All four reservoirs are approximately the same size. The turbines at all four sites have approximately the same hydraulic capacity so that drawdown at all four dams would not be limited by turbine capacity, but rather by the limiting drawdown rate of 0.6 m (2 feet) per day. This rate was established based on the results of the *1992 Reservoir Drawdown Test, Lower Granite and Little Goose Dams* (Corps 1993). At this rate, it would take approximately 40 days to lower the reservoirs from the top of normal pool to the

top of the temporary cofferdams. Breaching and removal of the cofferdams must be done to establish a free flowing channel.

The two means of passing flow for reservoir drawdown are: 1) through the spillway, and 2) through the powerhouse turbines. Since turbine hydraulic capacity is much greater than average mean daily flow from August through January, it is likely that the turbines would be used for most of the drawdown unless an unusually high inflow occurs. For unusually high flows, the spillway bays would be used for drawdown until the water surface reaches the ogee crest. Thereafter, powerhouse turbines would be used to draw the reservoir down at 0.6 m (2 feet) per day and to control flows. Figure B5 provides a summary hydrograph for the Snake River.

When the reservoir has been drawn down only 9.1 m (30 feet), more than 60 percent of the upstream reservoir shore would be exposed or above water, thereby significantly reducing the amount of shore line susceptible to damage from a breach caused by overtopping.

B.3.1 Risk of Embankment Overtopping During Excavation

One way to reduce risk of embankment overtopping during construction is to schedule embankment removal during a low flow period. Beginning construction in August to November produces the least risk. January through March is the period most susceptible to higher flows and thus has the highest risk of overtopping. Starting drawdown on August 1 is the most beneficial time to assure the embankments would not be overtopped due to sudden high flows during the construction period.

A freeboard of 3 m (10 feet) has been assumed for this study. (Freeboard is the distance the water surface is maintained below the top of excavation.) Another way to reduce the risk of overtopping during excavation is to vary the freeboard during drawdown from 3 to 6 m (10 to 20 feet), depending on whether excavation starts in August or December, respectively. A third way to reduce the risk of overtopping would be to increase freeboard proportionally from 3 to 6 meters as the top of excavation is lowered from the top of dam to the top of cofferdam.

The highest average mean daily flows from August through January are less than 1,700 m^3/s (60,000 cfs). The six turbines at any site can pass a combined flow of 2,230 m^3/s (78,700 cfs) with the minimum operating head of 6.1 m (20 feet), except at Ice Harbor for which the combined flow is approximately 1,784 m^3/s (63,000 cfs). Thus, if excavation is completed before January, there should be little risk of overtopping the embankments or cofferdams.

If river flows exceed 1,700 m^3/s (60,000 cfs) when the embankment has been excavated to within 9.1 m (30 feet) of tailwater, the embankment could overtop. This flow has a monthly frequency that exceeds the normal flow by approximately 30 percent in February and 40 percent in March. However, with average river inflow, the embankment excavations would not be overtopped during March, even at the lowest reservoir level.

To further reduce the risk of overtopping during excavation, the embankment could be removed in an asymmetrical pattern, so that the upstream face is left high while the downstream face is excavated at a faster rate. This sequence was used for the original embankment's construction. It keeps the crest high, but allows excavation to proceed faster than the reservoir drawdown rate. This excavation sequence would allow some protection against minor rises in river water due to inflows higher than 1,700 m^3/s (60,000 cfs). An estimate of the extent of damage that would be caused by rapid drawdown of the reservoir faster than the 0.6 m (2 feet) per day rate is contained in Annex H.

If higher than expected flows were to occur, close monitoring of river flows would be required in order to provide contractors with sufficient lead time to pull out of the construction area and avoid loss of equipment or lives.

B.4 Bank Protection

The purpose of bank protection is to prevent scour of the new channel bank or shore when subjected to velocities resulting from flows up to 11,890 m^3/s (420,000 cfs) after the embankment dam has been excavated.

Allowing the banks to erode naturally, except for embankments that support structures, is generally considered favorable; therefore, riprap would be applied only to bank areas that support existing roads, railroads, or other essential structures.

Typical forms of bank protection are as follows:

- Riprap
- Jute matting
- Concrete filled fabri-form
- Hand-placed stone
- End-dumped stone
- Concrete paving.

For this study, riprap was chosen as the primary form of bank protection. Riprap would be sized according to standard riprap design procedures, with a minimum diameter of 0.3 m (1.0 feet).

Material exposed by bank excavation would be sandy gravel, cobbles, and occasional boulders on the right bank, and rock that does not need protection in the main channel. Areas that are to receive bank protection are shown in Figures B2 through B4. Bank protection has been designed to remain functional for velocities resulting from flows of 11,890 m^3/s (420,000 cfs).

B.5 Navigation Lock and Spillway Modifications

Modifications to the navigation locks and spillways to provide additional channel width are not necessary to achieve acceptable fish migration velocities with the use of fish passage features at any of the four dams. All flows would pass around the remaining concrete structures and through the new channels.

Hydraulic studies have shown that even with the navigation locks used as a supplemental flow passage, fish passage features would still be needed because minimum velocities would exceed 1.5 mps (5 fps). Increasing the number of fish passage features is far less costly than physically modifying the navigation lock or spillway as a means of providing acceptable velocities.

In addition, modification of the spillway, specifically for cofferdams and construction sequencing, would take significantly longer than the 6-month construction window allocated for the embankment excavation and river channelization activities.

B.6 Construction Methodology

B.6.1 General

The construction methodology used for this study assumes average achievable excavation rates and reasonably common construction methods employing conventional construction equipment. The

construction period is primarily limited by the drawdown rate of 0.6 m (2 feet) per day and the sequencing of construction activities.

Other construction techniques could be employed to increase production rates, but the construction window and the drawdown rates would still govern.

B.6.2 Excavating Unit Systems

Excavation is assumed to be accomplished by various numbers of excavating unit systems, defined as a labor crew and equipment needed to excavate, transport, and stockpile materials.

Each excavating unit system consists of: a prime excavator; one or more dozers; a sufficient number of trucks to keep the excavator continuously productive; and other support equipment and labor to maintain haul roads, direct traffic, and handle material at stockpiles. The number of trucks required at each site is a function of the round trip haul time. At the stockpile area, two track dozers and compactors are used to spread the deposited material, and a grader and water truck are used for road maintenance.

B.6.3 Excavation Equipment

Excavation, loading, hauling, and stockpiling of the excavated embankment material requires very large units to perform efficient cycling times for handling materials. For the embankment excavation, scrapers cannot be used, but instead, large hydraulic excavators (with backhoe attachment) loading large-capacity; end-dump trucks on the excavated surface would be more efficient. This system could also be used for excavation of the dam foundation between the embankment cofferdams. An alternative approach would be to use large-capacity, dual engine, self-loading scrapers to load and haul and place soil materials, but this would require a larger working area and could be less effective if adequate turnaround areas are not available.

Typical excavating equipment and production rates are shown in Table B2.

Table B2. Typical Excavation Equipment and Production Rates

Name	Model	Capacity	Typical Rate
Large Hydraulic Excavator (general)	CAT 5130	9.9 m^3 (13 cy) bucket	1150 m^3/hr (1500 cy/hr)
Small Hydraulic Excavator (riprap)	CAT235D	3.8 m^3 (5 cy) bucket	153 m^3 (200 cy/hr)
Front End Loader (general)	CAT 992D	10.7 m^3 (14 cy) bucket	917 m^3/hr (1200 cy/hr)
Hauling Trucks	CAT 777c	81.6 metric ton (90 ton)	46 m^3 (60 cy)/hr
Dozer	CAT D-9N or D-7H		
Grader	CAT 16-G		
Compactor	CS563		
Dragline	American 12220	7.6 m^3 (10 cy)	321 m^3/hr (420 cy/hr)

All above water excavation is assumed to be performed by track mounted hydraulic excavators with 9.9-m^3 (13-cy) buckets and/or rubber-tired front-end loaders with 10.7-m^3 (14-cy) buckets. Trucks are assumed to be 81.6-metric-ton (90-ton), 46-m^3 (60-cy) end-dump, off-highway haul units. With advance notice of bidding, these types of equipment are commonly available in the Pacific Northwest and western United States, and in numbers that would permit removal of all four dams simultaneously. Uniform bucket fill factors and job efficiency factors were developed for establishing production rates.

B.6.4 Excavation Procedures/Rates

The dam embankment excavation rate is a function of the following:

- The size and type of equipment
- The type of material
- The space available for turnaround of haul trucks.

Equipment cannot efficiently travel on the riprap and rockfill surface areas. The upstream and downstream width of rockfill and riprap is 1.5 m and 1.8 m (5 feet and 6 feet), respectively, for a total of 3.4 m (11 feet); therefore, at any given embankment excavation surface level, 3.4 m (11 feet) of width would be unusable as a roadway. Also the core material on the upstream side might be wet and would be more difficult to work on than the gravel fill zones and filters.

Front end loaders would be used in matching numbers to track-mounted hydraulic excavators because front end loaders are generally less expensive to transport and assemble, but less versatile in dealing with variable excavation conditions.

Until the working surface is 30.5 m (100 feet) wide, there would be room for only one excavating unit. A single, track-mounted, hydraulic excavator would be used because it has greater flexibility in excavating all materials. Its initial production rate was assumed to be reduced to 765 m^3/hr (1,000 cy/hr) from a normal rate of 1,150 m^3/hr (1,500 cy/hr). Where multiple sets of excavating unit systems are to be used, each additional excavating unit was assumed to require an additional 15.2 m (50 feet) of working surface width. Excavating units could be staggered along the embankment's length, and the study team assumed trucks would share a common turnaround area. A summary of excavation rates versus available surface width on the dam is shown in Table B3. Figures B6 through B9 graphically show the drawdown elevation and the corresponding embankment excavation during the period of embankment excavation.

Table B3. Number of Excavation Units Vs. Available Space

Depth Below Top of Dam (m)	Depth Below Top of Dam (feet)	Surface Width (m)	Surface Width (feet)	Number of Excavating Units	Anticipated Production Rate (m^3/hr)	Anticipated Production Rate (cy/hr)	Total (m^3/hr)	Total (cy/hr)
0 - 5.0	0 - 16.7	13.7 - 33.8	45 - 111	1 Hydraulic Excavator	764	1,000	764	1,000
5.0 - 8.7	16.5 - 28.5	33.8 - 48.5	111 - 159	1 Hydraulic Excavator	1,147	1,500	1,147	1,500
8.7 - 12.3	28.5 - 40.0	48.5 - 63.0	159 - 206	1 Hydraulic Excavator	1,147	1,500		
				1 Front End Loader	917	1,200	2,064	2,700
12.3 - 16.6	40.0 -54.4	63.0 – 80.0	206 - 263	2 Hydraulic Excavators	2,293	3,000		
				1 Front End Loader	917	1,200	3,211	4,200
Below 16.6	Below 54.4	80+	263+	2 Hydraulic Excavators	2,293	3,000		
				2 Front End Loaders	1,835	2,400	4,128	5,400

B.6.5 Embankment Material Stockpiling

The embankment material would be removed to stockpiles located on the same side of the river as the new channel and within 1.6 km to 3.2 km (1-to-2 miles) of the embankments. The embankments consist of several zones of material, but separation of each type would severely increase costs and limit excavation rates because of the material zone configuration and the thin layers of filter and riprap material. Material would be removed in two classifications comprised of: 1) impervious core, and 2) all other materials, consisting primarily of gravel fill plus filter material. Riprap would be stockpiled to the extent segregation is possible. Obtaining new filter materials would be cheaper than separating and stockpiling it separately for future use. Stockpiling of impervious core material separately is important because it could be used for topsoil to landscape the channels for plantings of trees, shrubs, and grass. Figures B10 through B13 show anticipated material stockpile areas and potential hauls roads.

B.6.6 Embankment Excavation Operations

Excavation operations are assumed to involve two 10-hour shifts per day in a 5-day work week (Saturday and Sunday off). At the maximum embankment excavation rates and assuming 100 hours/week (20 hours/day times 5 days), the embankment and abutments at all dams could be removed at the schedule shown in Table B4. In order to provide additional time to deal with contingencies that may arise, an addition two shifts per week is added to the excavation schedule.

Since maximum production rates of excavation indicate that the top of the embankment dam could be lowered much faster than the reservoir drawdown rate of 0.6 m (feet) per day, the drawdown rate is the governing factor in the construction schedule. For the contractor to maximize the efficiency of the excavating equipment, it is likely that the actual start of excavation would be delayed approximately 4 to 5 weeks after the beginning of the August 1 drawdown. This procedure will also provide greater freeboard for a longer period of time, thus reducing the risk of overtopping should problems develop during the early stages of drawdown.

The dates in Table B4 are for excavation of the embankment materials. Subsequent activities for breaching and removal of the cofferdams are not shown in Table B4.

Table B4. Embankment Excavation Time at Maximum Excavation Rate (Drawdown Begins August 1)

Dam	Excavation Duration	Excavation Begins	Excavation Ends
Lower Granite	28	August 30	September 28
Little Goose	21	September 7	September 28
Lower Monumental	55	August 1	September 25
Ice Harbor	61	August 2	October 2

The most apparent problem may be from the presence of saturated materials in the embankment. Excavating, hauling, and stockpiling these materials may be problematic because of possible difficulties in handling the material and driving vehicles across existing and new fill sections. It is not possible to drain the reservoir the required time in advance of drawdown for the embankment materials to drain to a point where these problems are eliminated.

The outer shells of the embankments would remain in place while excavation below water surface elevation proceeded within the cofferdam area. Excavating this interior under relatively dry conditions would be done using the ongoing excavation processes. This procedure minimizes the volume of material that must be excavated "in the wet" using underwater methods such as crane-mounted draglines. In-water

excavation requires draglines to operate at a much slower production rate. The area within the cofferdams would require two temporary pump stations – one upstream of the core and one downstream of the core – to allow dry excavation operation.

It is very likely that the downstream cofferdams at Lower Granite and Little Goose Dams and the abutment cofferdams at Lower Monumental and Ice Harbor Dams will not be sufficiently impervious to allow dry construction operations. Placement of silt core material on these water-side cofferdam surfaces is proposed to make these structures relatively impervious so excavation may proceed.

B.6.7 Embankment Removal Sequence for Lower Granite and Little Goose Dams

The embankment material could be removed to the top of the original embankment's upstream and downstream cofferdams. The two cofferdams would remain in place while the saturated embankment material between them would be excavated to the foundation using surface excavation equipment. Figure B14 illustrates the embankment cross section and the phasing of excavation equipment for these dams.

The area between cofferdams should remain accessible as the upstream cofferdam has an impervious blanket on the upstream face. The impervious blanket was removed from the downstream face of the downstream cofferdam, but the cofferdam fill material should be sufficiently compacted from construction and the weight of the embankment fill above to minimize leakage. Pumps would handle leakage.

B.6.8 Embankment Removal Sequence for Lower Monumental and Ice Harbor:

The abutment material for the new channel downstream of the existing embankment could be excavated at an unrestrained rate. The embankment would be left in place while the reservoir is being drawn down. After material downstream of the existing embankment is excavated, equipment would move upstream to the embankment and abutment material would be excavated at an unrestrained rate down to the new channel's invert. Temporary cofferdams both upstream and downstream would be left in place parallel with the shore line at the outer edge of the required excavation, behind which excavation of abutment material could proceed to the channel invert using surface excavation equipment. Pumps would handle seepage. The remaining cofferdam and any river bottom material would then be excavated by dragline.

Embankment removal is limited by the reservoir drawdown rate of 0.6 m (2 feet) per day. However, with proper sequencing, most excavation could be completed at an unrestrained rate. Proper sequencing would involve starting excavation several weeks after drawdown begins and leaving a natural cofferdam of existing earth in place between the river and the excavation area, behind which excavation could be made in the dry. Figure B15 illustrates the embankment in elevation view and shows where the temporary earth dike would be used for these dams. Note that the Ice Harbor configuration is opposite hand.

B.6.9 Cofferdam Breaching and Removal Sequence

The original embankments for Lower Granite and Little Goose dams were constructed between two parallel cofferdams that were incorporated into the heel and toe of the dam. While the embankment material is being removed from between these cofferdams, a vertical row of sheet piles would be driven in and perpendicular to the cofferdams about 30 m (98 feet) from the navigation lock wall. A typical construction sequence for Lower Granite and Little Goose Dams is shown in Figure B16. The sequence of cofferdam breaching is illustrated in the photographs in Figures B18 and B19.

This construction sequence would allow the cofferdams to be breached while reducing the possibility of losing equipment. The cofferdams are primarily constructed of rockfill; however, very large rock next to

the navigation lock wall was used in the upstream cofferdam for closure against the final flows. This material would be more difficult to remove than the cofferdam material and may require special handling. The downstream cofferdam had an impervious blanket on the downstream face. Some of it was removed prior to placing the remaining embankment fill between the cofferdams in order to allow a free flow path through the lower part of the embankment.

At Lower Monumental and Ice Harbor dams, temporary earth dikes would be left in place upstream and downstream. These temporary cofferdams would consist of unexcavated abutment material and would be located at the riverside limit of the abutment excavation. Sheetpiles would be placed 30 m (98 feet) from the navigation-lock end, driven into and perpendicular to the temporary cofferdams. Sheetpiles would be used to control erosion during breaching. A typical construction sequence for Lower Monumental and Ice Harbor Dams is shown in Figure B17.

Cofferdam removal would be similar at all four dams. Material would be removed from the downstream cofferdam between the sheetpiles and the outer end, allowing water to flow into the area between the two cofferdams. The upstream cofferdam would be breached, allowing the water surfaces upstream and downstream to equalize. When the water surfaces equalize, the downstream cofferdam would be removed. If dams are not removed sequentially, then downstream reservoirs could be lowered to facilitate removal of the cofferdams. Once the water surface elevations on both sides of the upstream cofferdam are equalized, the removal of both cofferdams could begin. Excavation would commence at the cofferdam breach by removal of the sheet piles and proceed toward the shoreline. This is a controlled breach scenario and assumes river flows of 1,700 m^3/s (60,000 cfs) or less. Cofferdam material above water would be removed with a hydraulic excavator, while material below water would be removed by a dragline.

If higher river flows occurred, the head differential might result in velocities that preclude controlled breach and removal of the cofferdam. In this scenario, excavation of the cofferdams would commence from the shore, providing a water passage that would force an uncontrolled breach. In this case, the river flow would carry away the cofferdam material. The amount of material carried away would depend on flows. Subsequent high river flows through the new channel would continue to carry the material further downstream.

The total volume of material in the cofferdams at all four sites is 1,030,400 m^3. Less than 40 percent of this material consists of silt-sized particles. The bulk of the material is sand and gravel. Under expected conditions, less than 10 percent of the material in the cofferdams would not be recovered from the river by dragline excavation. However, extreme flow events could lead to a worst-case scenario where up to 90 percent of the silt is lost to the river. This range of silt addition to the river is considered a very small fraction of the volume of sediment that will be mobilized immediately following breaching the cofferdams. Specific information on sediment concentrations is not yet available.

There would be some additional sediment introduced into the river during removal of cofferdams and during levee construction. In addition, Lower Monumental and Ice Harbor dams together would require approximately 142,400 m^3 of local dredging of the riverbed material to produce acceptable velocities in the new channel.

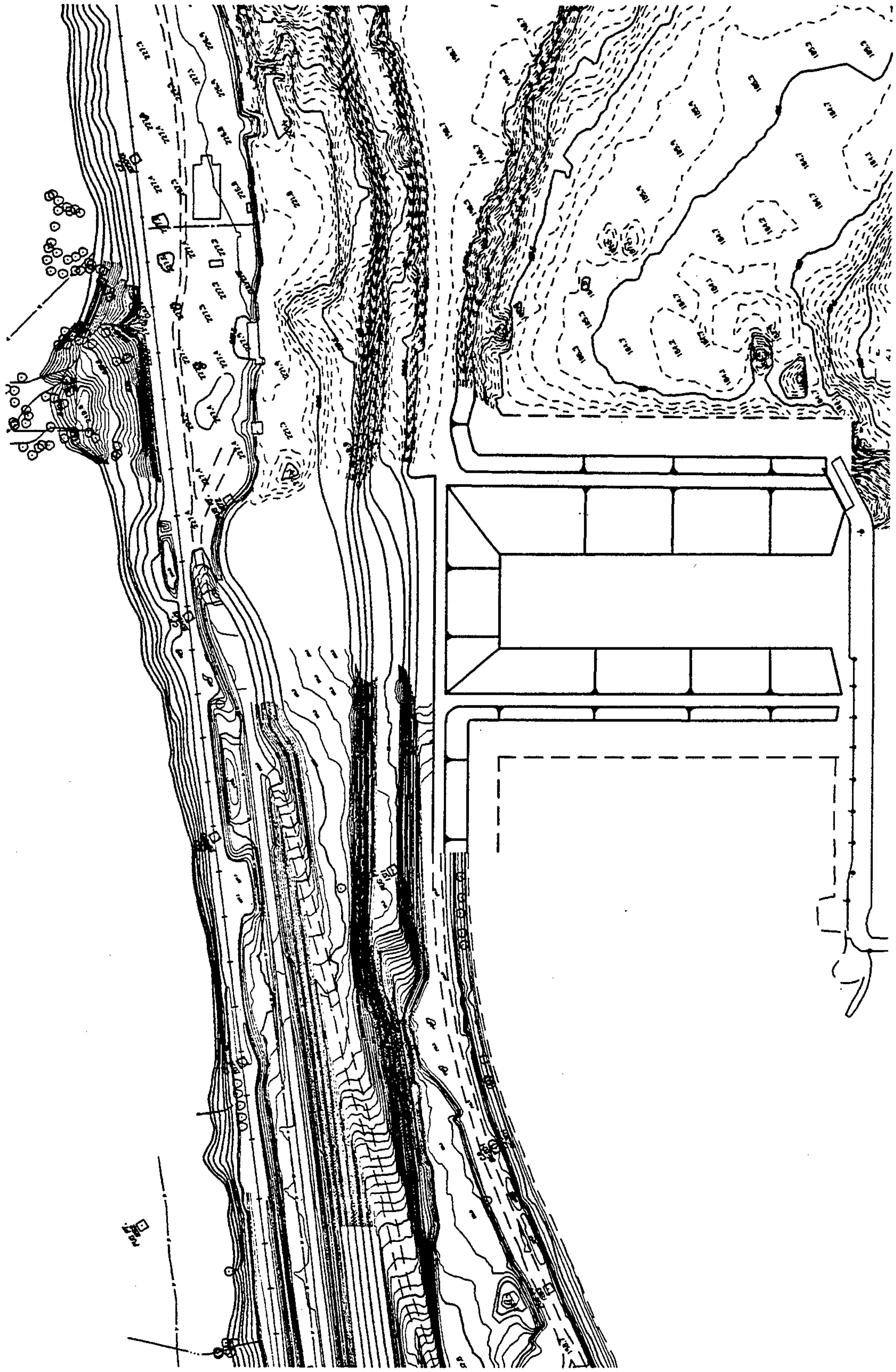
Extremely close coordination and cooperation would be required to ensure that controlled breaching can be achieved safely at all of the dams. Contracts would need to be structured such that constraints on the individual contractors prevent delays, contractual claims and increased costs.

B.6.10 Cofferdam Removal Equipment

Cofferdam removal would be accomplished by a hydraulic excavator for material above the water surface and by dragline for material below the water surface. Estimates for quantities of material to be removed above and below water were based on the water surface elevation for a flow of 1,700 m^3/sec (60,000 cfs) at each site. The hydraulic excavator is assumed to be a 7.6-m^3 (10-cy) CAT 5130 with a productivity rate 50 percent of normal for this equipment or approximately 573 m^3/hr (750 cy/hr). The dragline would have a 7.5-m^3 (10-cy) bucket and a productivity rate of 321 m^3/hr (420 cy/hr). End-dump trucks would need to back up along the cofferdam crest to receive material.

B.6.11 Sequenced Removal of the Four Lower Snake River Dams

Removal of all four lower Snake River dams can be sequenced in several ways. The proposed method selected as a result of this study is to sequence the work so that Lower Granite and Little Goose Dams are breached during the fifth year of the construction period. Lower Monumental and Ice Harbor would be breached during the sixth year of the construction period. Several other variations are possible, however this method provides a realistic phasing of design and construction activities and is not overly optimistic for this level of feasibility. See Annex W for more detailed discussion of construction sequencing.



US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER GRANITE DAM - GENERAL EMBANKMENT REMOVAL CONFIGURATION

Figure:
B1

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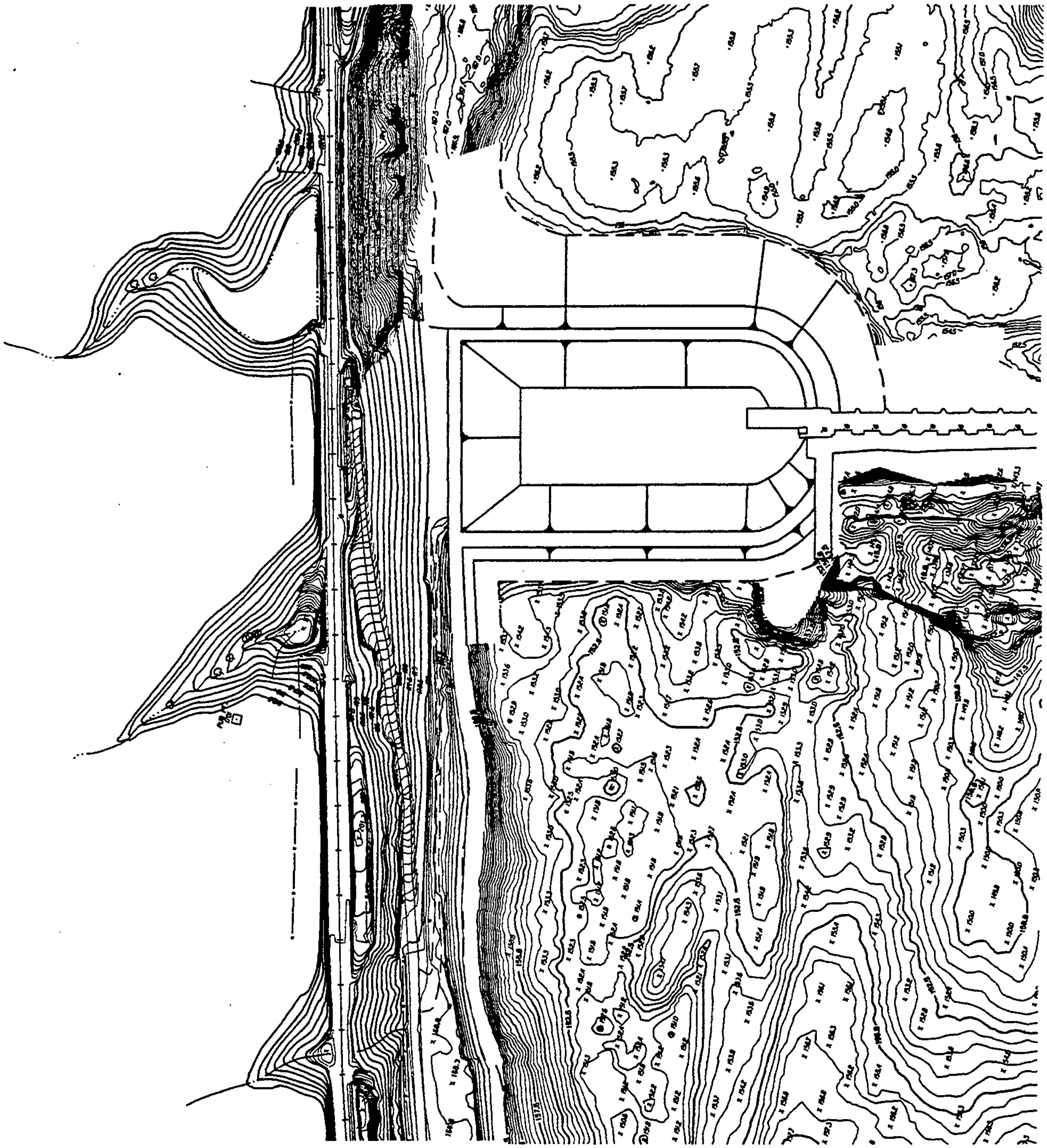


US Army Corps of Engineers
Walla Walla District

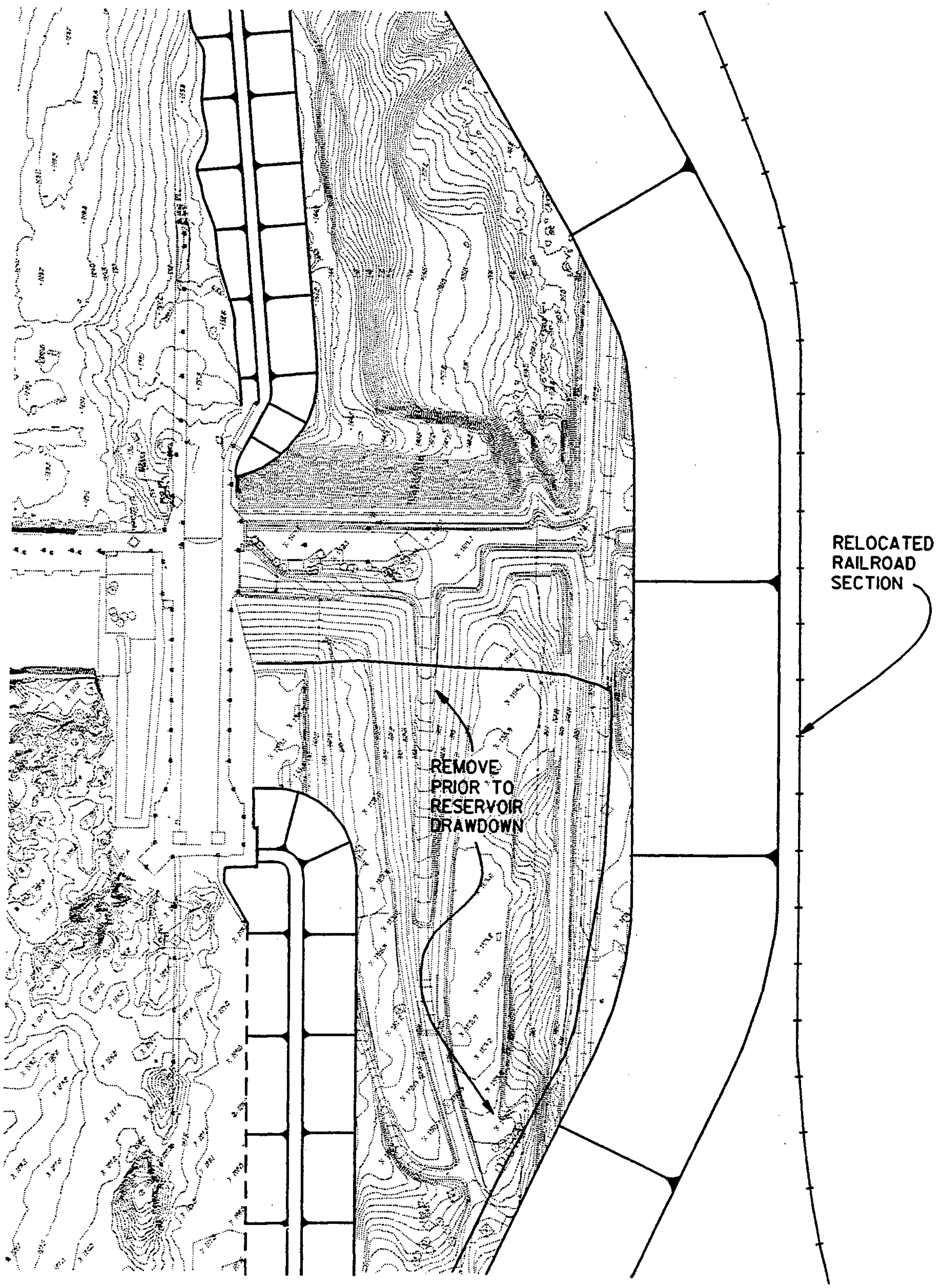
LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LITTLE GOOSE DAM - GENERAL EMBANKMENT REMOVAL CONFIGURATION

Figure: B2

CADD FILENAME: E:\drawdown\plates\b\pltb2.dgn

PLOT TIME: 25-OCT-1999 07:41



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER MONUMENTAL DAM - GENERAL EMBANKMENT REMOVAL CONFIGURATION

Figure:
B3

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ICE HARBOR DAM - GENERAL EMBANKMENT REMOVAL CONFIGURATION

Figure: B4

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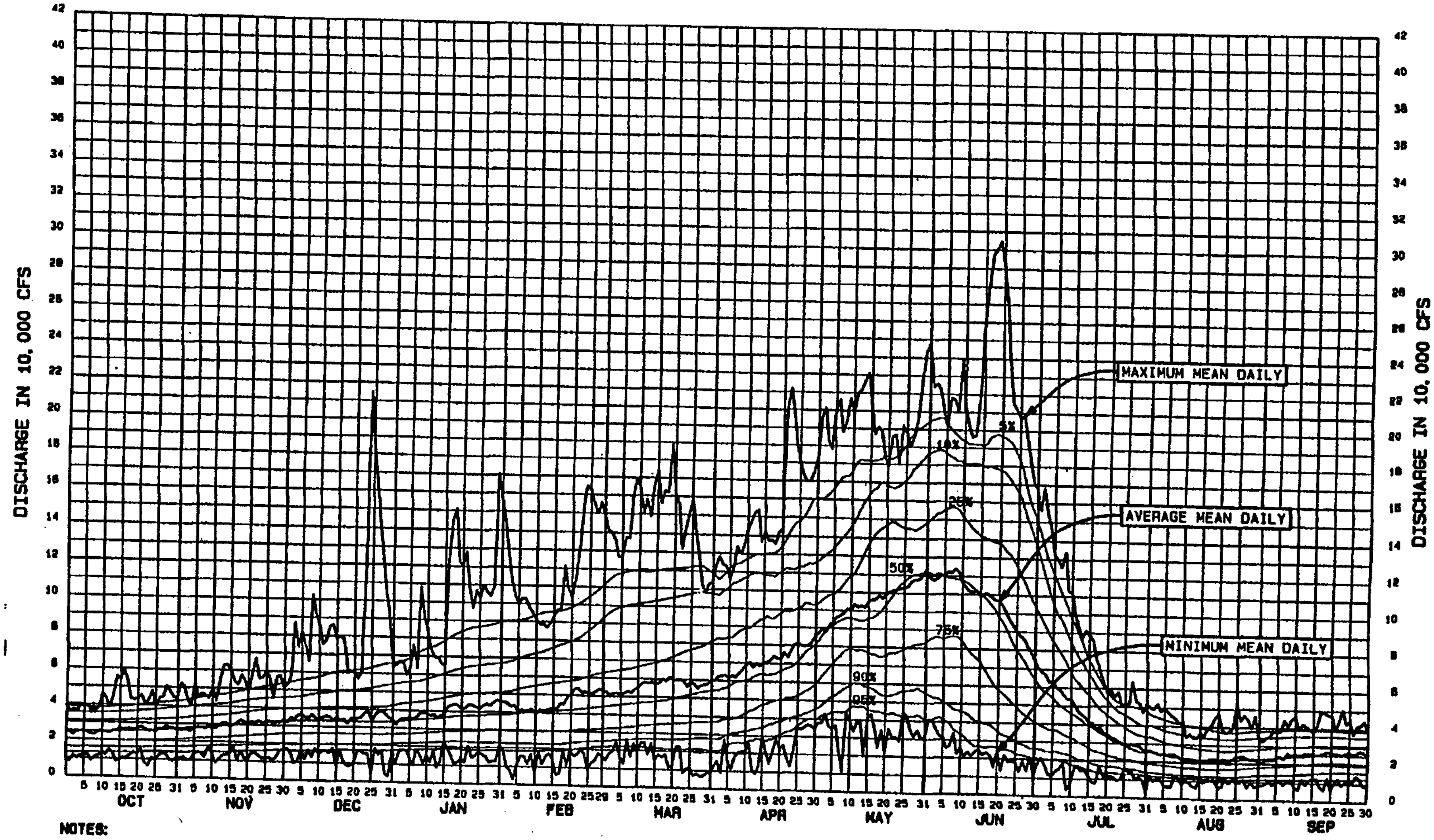

NOTES:

1. SUMMARY HYDROGRAPHS PLOTTED FROM CORPS OF ENGINEERS MEAN DAILY OUTFLOW DATA FOR ICE HARBOR DAM.
2. PERIOD OF RECORD IS OCTOBER 1962 THROUGH SEPTEMBER 1995.
3. DRAINAGE AREA AT ICE HARBOR DAM IS 108,500 SQUARE MILES (APPROXIMATELY).
4. EXCEEDENCE LINES REPRESENT THE PERCENTAGE OF TIME THE FLOW IS EQUALLED OR EXCEEDED ON THAT PARTICULAR DAY.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
EMBANKMENT EXCAVATION AND RIVER CHANNELIZATION
SUMMARY HYDROGRAPH

Figure: B5

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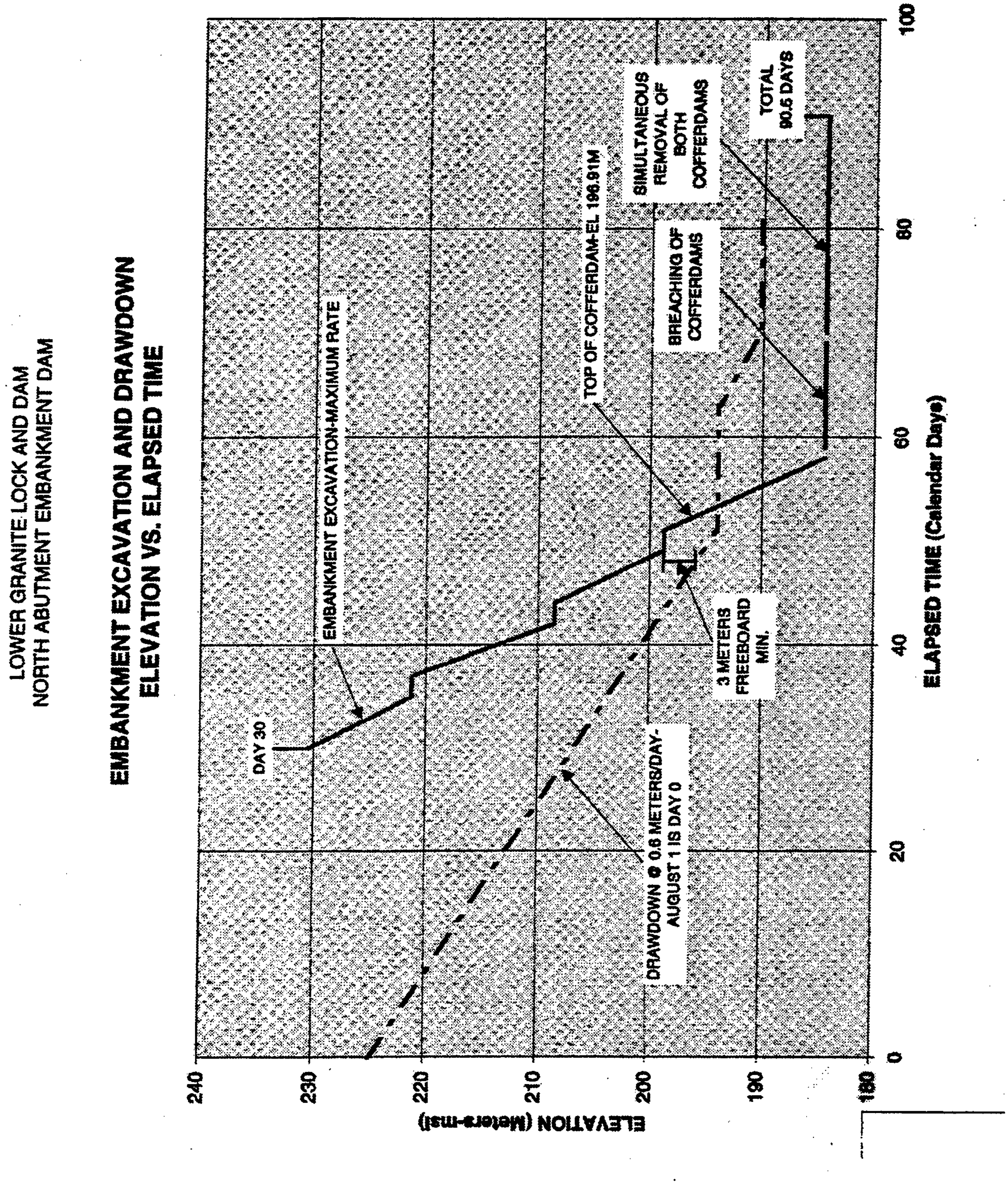

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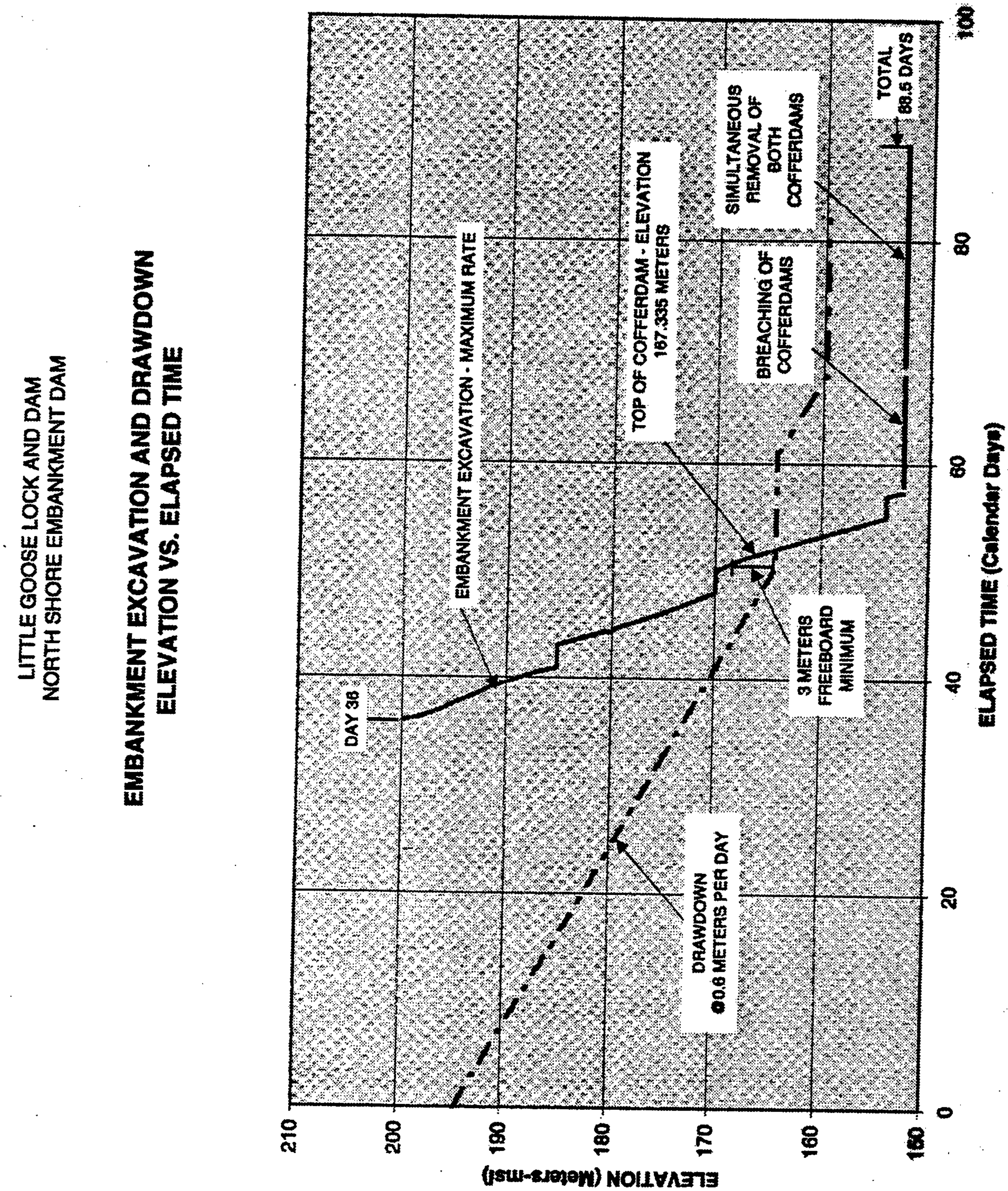
LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

EMBANKMENT EXCAVATION AND DRAWDOWN ELEVATION VS. ELAPSED TIME

Figure: B6

CADD FILENAME:E:\drawdown\plates\b\pltb6.dgn PLOT TIME:17-SEP-1999 07:36



US Army Corps of Engineers Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

EMBANKMENT EXCAVATION AND DRAWDOWN ELEVATION VS. ELAPSED TIME

Figure: B7

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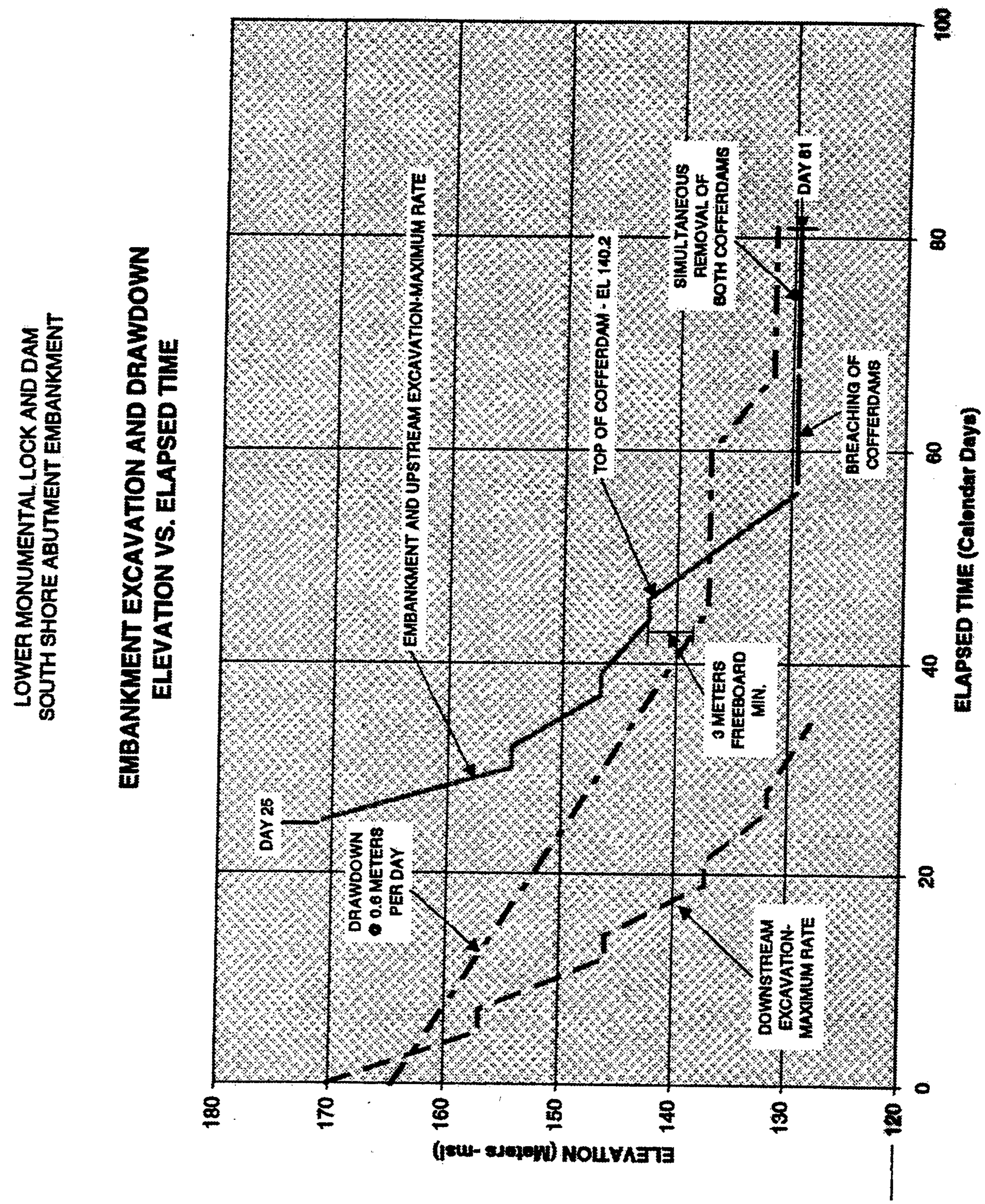

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

EMBANKMENT EXCAVATION AND DRAWDOWN ELEVATION VS. ELAPSED TIME

Figure: B8

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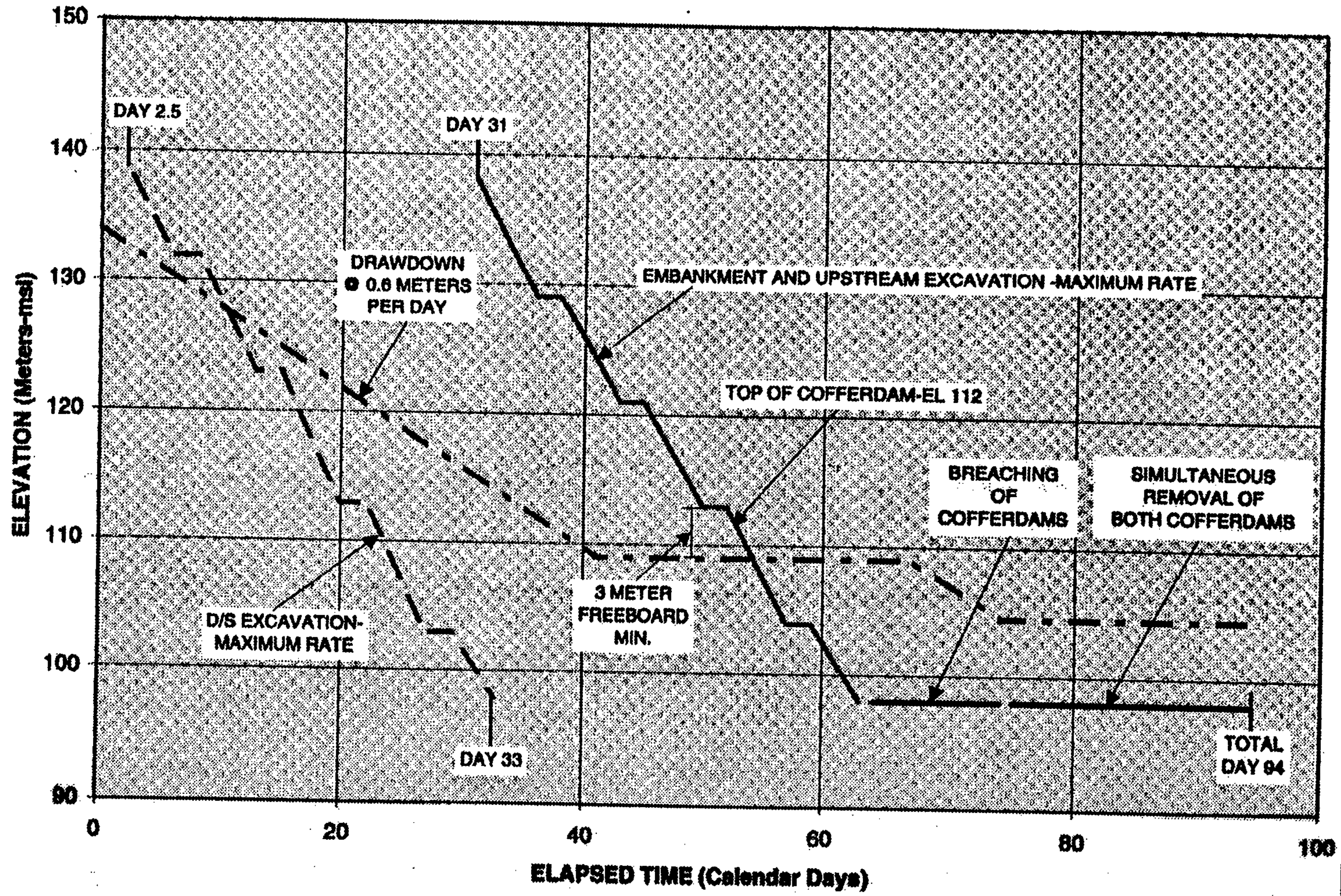
ICE HARBOR LOCK AND DAM
NORTH SHORE EMBANKMENT DAM

EMBANKMENT EXCAVATION AND DRAWDOWN ELEVATION VS. ELAPSED TIME



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

EMBANKMENT EXCAVATION AND DRAWDOWN ELEVATION VS. ELAPSED TIME

Figure: B9

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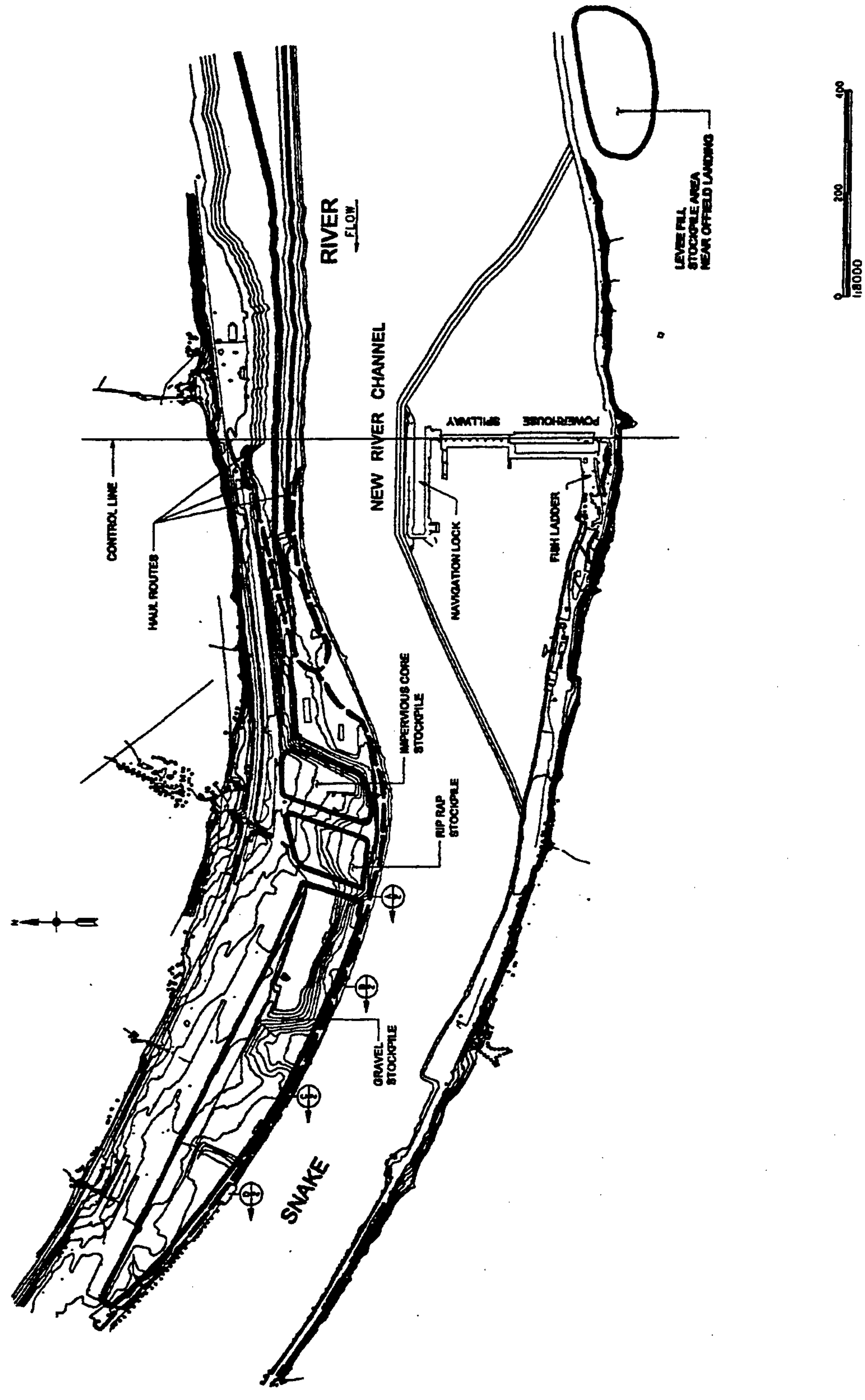
SHEET MAIN SCALE:



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER GRANITE STOCKPILE AREAS AND HAUL ROADS - PLAN

Figure: B10

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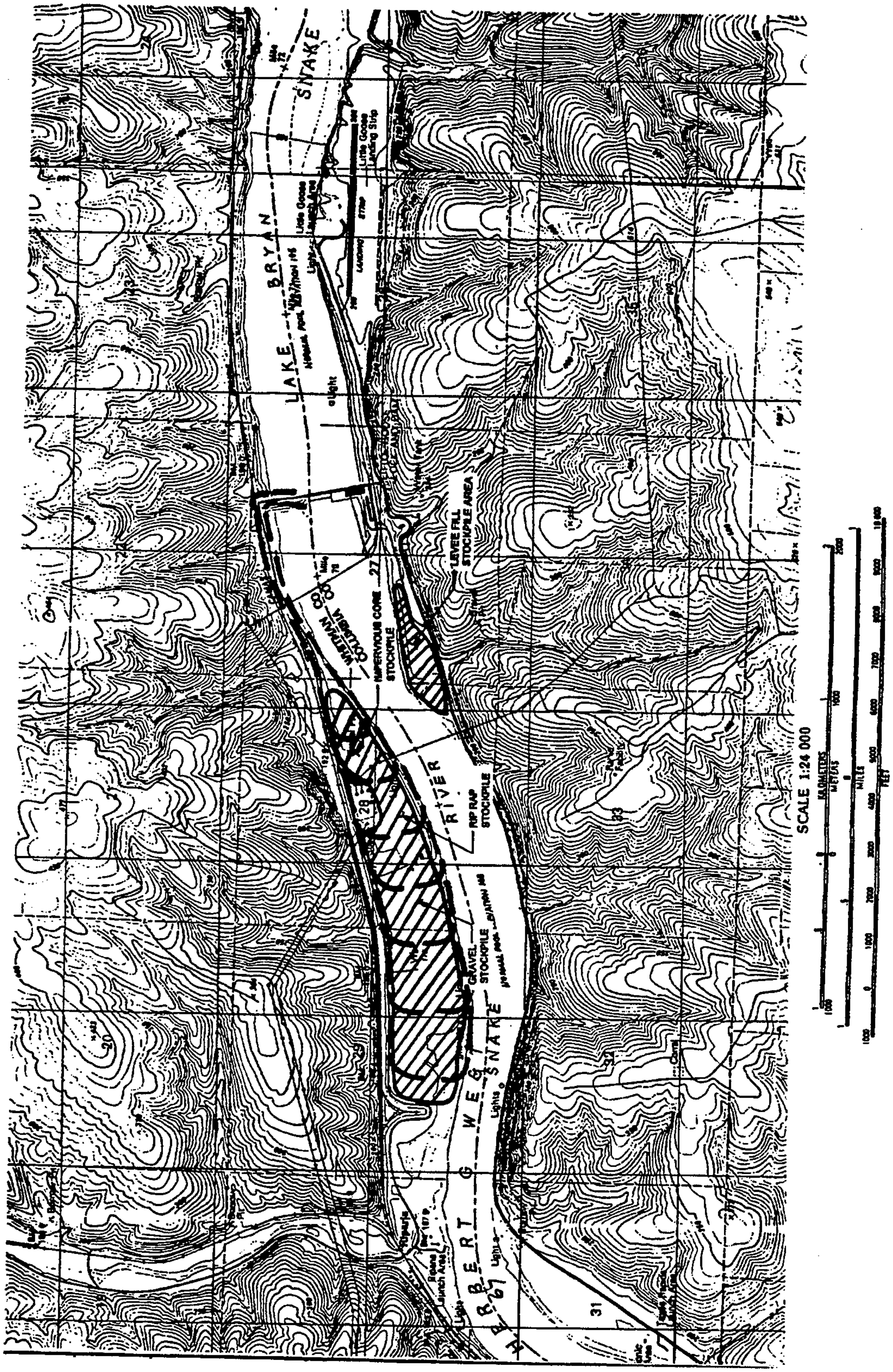
SHEET MAIN SCALE:



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Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LITTLE GOOSE STOCKPILE AREAS AND HAUL ROADS - PLAN

Figure: B11

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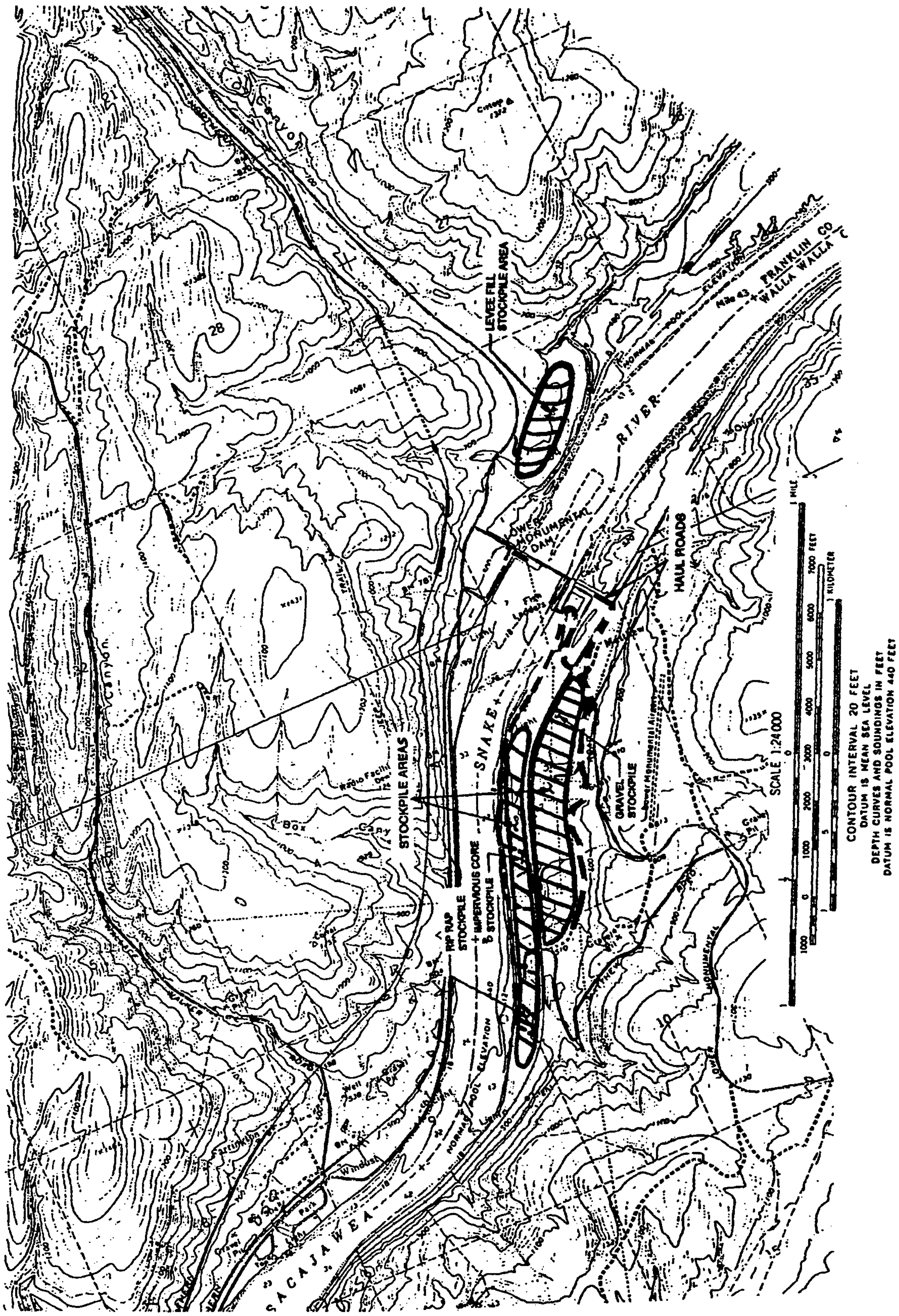

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER MONUMENTAL STOCKPILE AREAS AND HAUL ROADS - PLAN

Figure: B12

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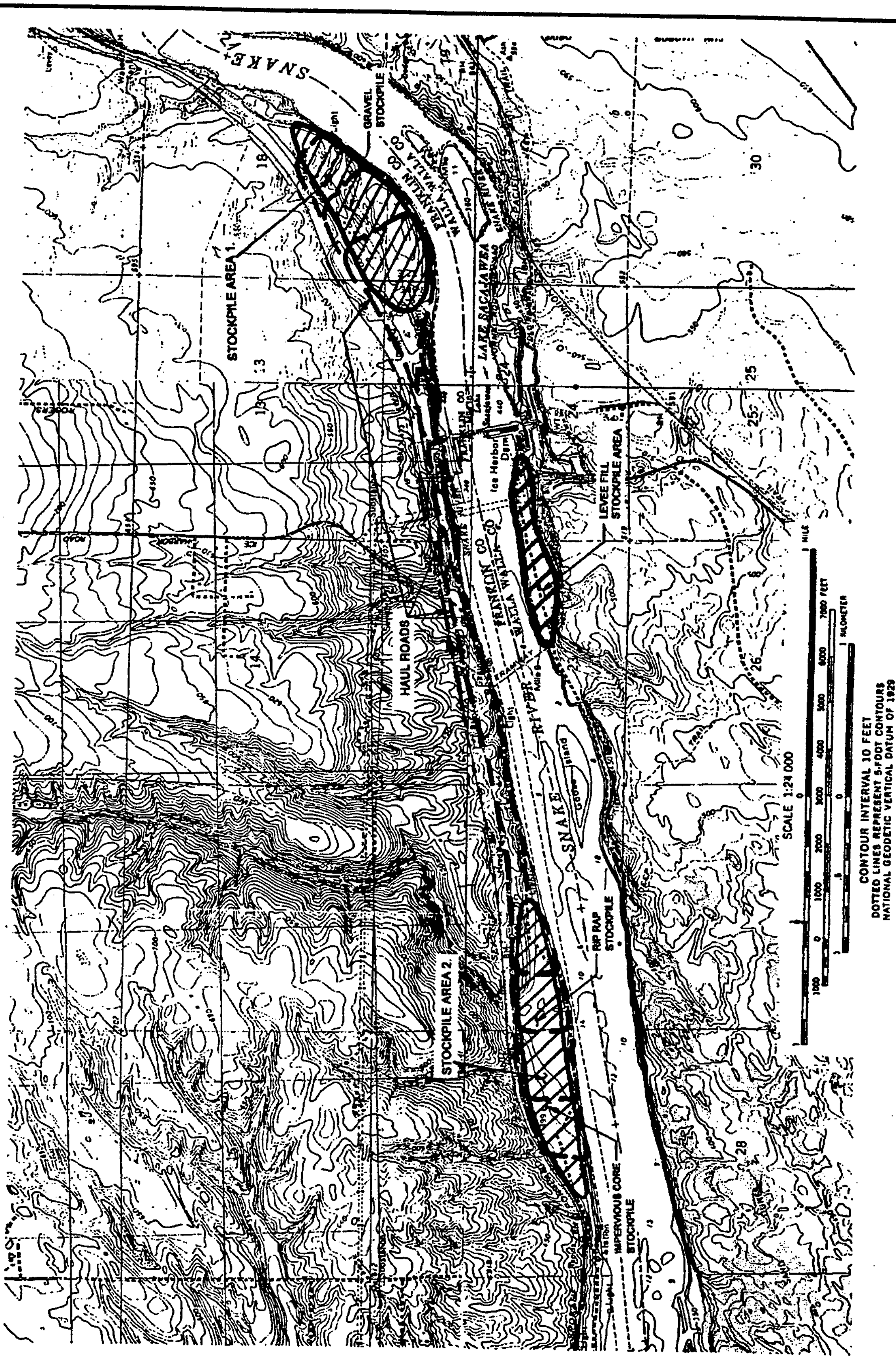

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ICE HARBOR STOCKPILE AREAS AND HAUL ROADS - PLAN

Figure: B13

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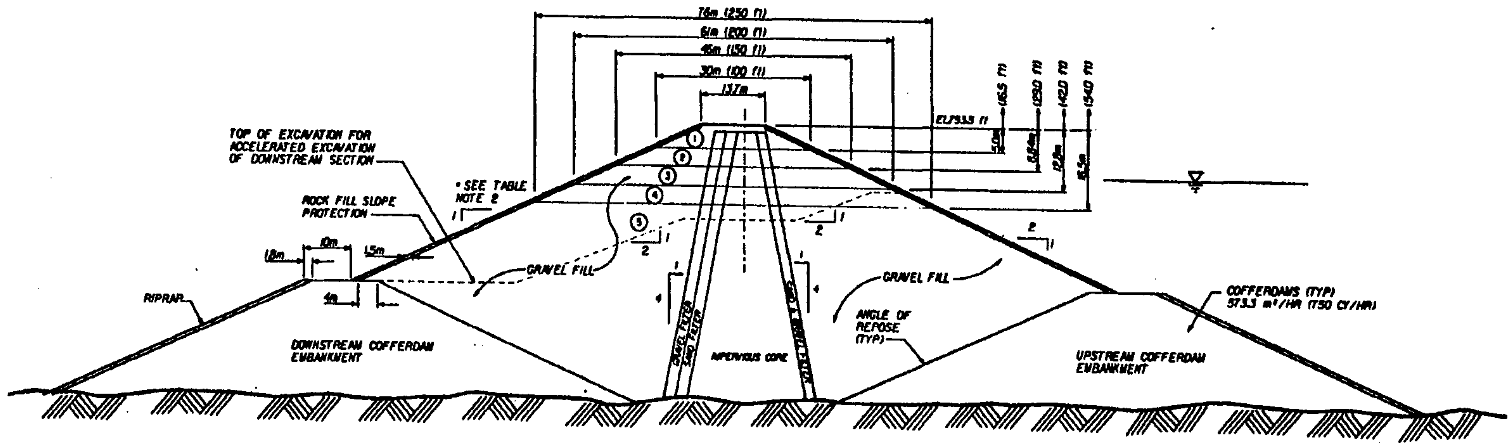

EXCAVATING EQUIPMENT & RATES

1. 1 EXCAVATOR @ 765m³/HR (1000/CY/HR)
2. 1 EXCAVATOR @ 1150m³/HR (1500/CY/HR)
3. 1 EXCAVATOR @ 1150m³/HR (1000/CY/HR) PLUS 1 FRONT END LOADER @ 920m³/HR (1200/CY/HR) • 2070m³/HR (2700/CY/HR)
4. 2 EXCAVATOR @ 1150m³/HR (3000/CY/HR) PLUS 1 FRONT END LOADER @ 980m³/HR (1200/CY/HR) • 3220m³/HR (4200/CY/HR)
5. 2 EXCAVATOR @ 765m³/HR (3000/CY/HR) PLUS 2 FONT END LOADER @ 1830m³/HR (2400/CY/HR) • 4130m³/HR (5700/CY/HR)

TYPICAL EMBANKMENT SECTION

0 1 2 3 4
1:100

NOTE:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN METERS.
2. EMBANKMENT SECTION SHOWN IS TYPICAL FOR EACH SITE WITH THE FOLLOWING EXCEPTIONS:

PROJECT SITE	FILL SLOPES		CREST WIDTH		CREST ELEVATION		AVERAGE HEIGHT	
	UPSTREAM	DOWNSTREAM	meter	(ft)	meter	(ft)	meter	(ft)
Lower Granite	2H:1V	2H:1V	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Little Goose	2.5H:1V	2H:1V	30	9.14	[illegible]	[illegible]	47.80	[illegible]
Lower Monumental	2H:1V	2H:1V	30	9.14	170.08	[illegible]	40.60	[illegible]
Ice Harbor	2H:1V	1.5H:1V	25	7.62	[illegible]	[illegible]	[illegible]	[illegible]

3. COFFERDAMS SHOWN ARE PRESENT AT LOWER GRANITE AND LITTLE GOOSE ONLY.
4. EMBANKMENTS FOR LOWER MONUMENTAL AND ICE HARBOR DO NOT HAVE COFFERDAMS.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
TYPICAL EMBANKMENT EXCAVATION AND TYPICAL CROSS SECTION
AT LOWER GRANITE AND LITTLE GOOSE

Figure: B14

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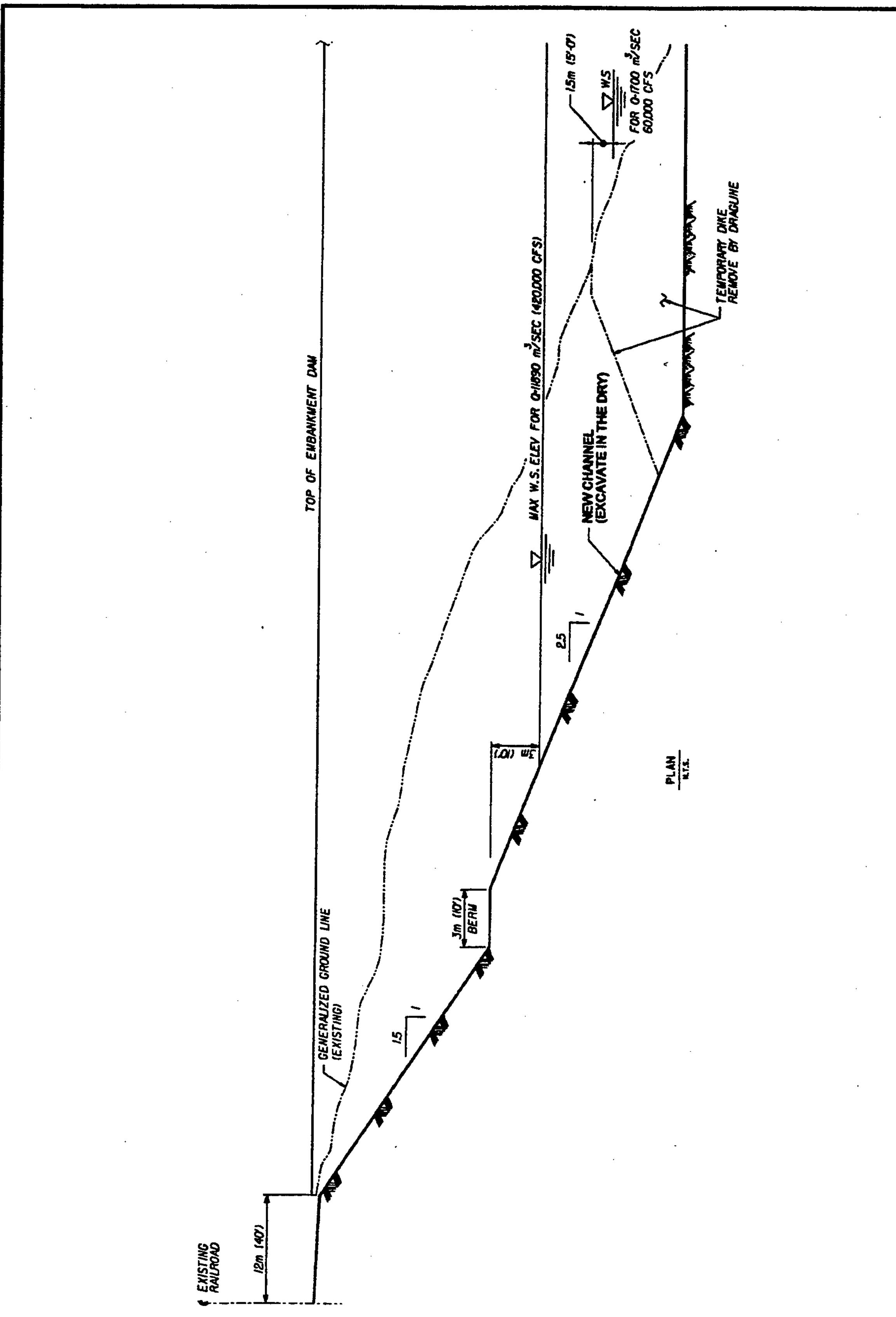

NOTES:

1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN METERS.
2. EXCAVATION SHOWN FOR SOUTH SHORE AT LOWER MONUMENTAL DAM. OPPOSITE HAND FOR NORTH SHORE EXCAVATION AT ICE HARBOR DAM.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TYPICAL ABUTMENT EXCAVATION AT LOWER MONUMENTAL DAM
ICE HARBOR (OPP HAND)

Figure: B15

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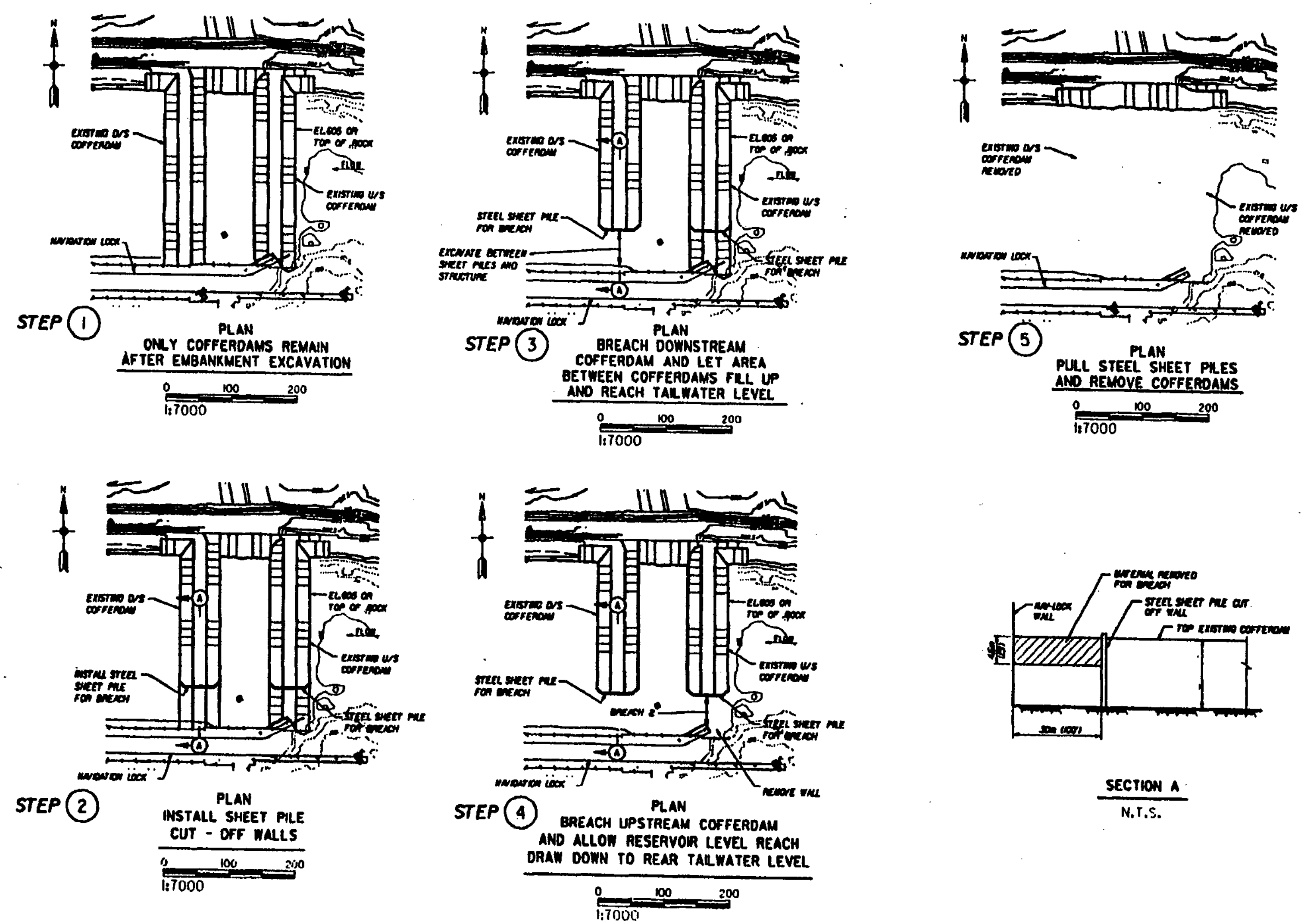

NOTES:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN METERS.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
TYPICAL COFFERDAM AND BREACH PLAN
LOWER GRANITE AND LITTLE GOOSE DAMS

Figure: B16

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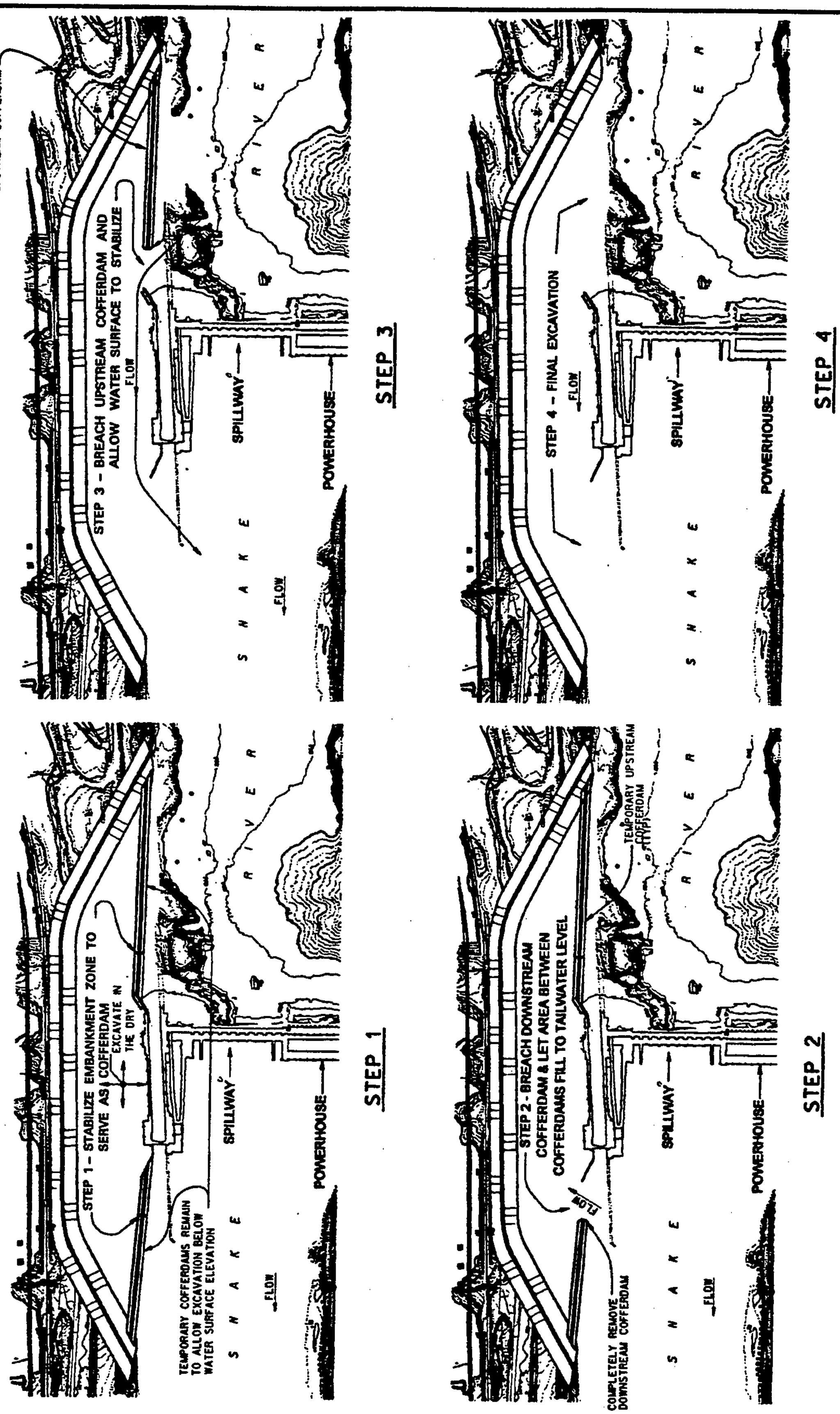

CONSTRUCTION SEQUENCE

NOTE: STEPS ILLUSTRATE WORK SEQUENCE FOR ICE HARBOR DAM. THE SAME SEQUENCE IS PLANNED FOR THE SOUTH SHORE OF LOWER MONUMENTAL DAM, (OPPHAND).

SHEET MAIN SCALE

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
TYPICAL EARTH DIKE AND BREACH PLAN
LOWER MONUMENTAL AND ICE HARBOR DAMS

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Annex C

Temporary Fish Passage Plan

Annex C: Temporary Fish Passage Plan

C.1 General

This report explores the range of options considered for maintaining upstream adult fish migration during the entire drawdown construction period and presents the recommended plan of action. Fish passage must continue during the drawdown process even though existing adult fish passage systems will become inoperable shortly after the drawdown process begins at each project. Therefore, temporary adult fish handling facilities must be developed, evaluated, and analyzed for the best alternative that will provide upstream migration.

This report covers conceptual and functional plans in sufficient detail to develop a cost estimate. The option selected for the action plan was based on the information available at this stage of concept development. The final decision concerning the preferred option will depend on results of further investigation into anticipated turbidity and suspended sediment levels during drawdown and it's effect on fish migration. Actual final design, schedule, installation procedures, and operating criteria may change during more detailed design at advanced stages of drawdown implementation.

Adult anadromous fish are in the river system at all times. Chinook and steelhead comprise the majority of migrating adults, although there are a limited number of other species including the endangered sockeye. Summer chinook migrate upstream from July to September. Fall chinook migrate from August to October. Late summer and late fall are periods when steelhead runs are the largest of the year, but there are a number of them migrating year round. From December through March, migration slows since fish tend to hold up when water temperatures are below 4.4 degrees Celsius (40 degrees Fahrenheit).

The period of drawdown construction occurs during a few upstream migrating fish runs containing large numbers of fish. These runs include endangered sockeye and threatened Snake River steelhead and fall chinook. Although steelhead can delay for up to two months without causing great impact to their migration, requiring all fish to delay during the period of construction for both drawdown seasons would greatly impact these and future fish runs. Consequently, adult passage must be maintained during this process.

In the early stages of each drawdown season, the forebay water surface elevation above each project involved in that drawdown season falls below minimum levels required by operating criteria for the fish handling facilities. Depending on river flow, tailwater elevations may fall below elevations required for the fish entrances and for attraction water pump systems to properly function. Worst case is that tailwater elevation would fall below existing minimum operating levels ranging from 2.9 meters (m) (9.5 feet [feet]) to 6.0 m (19.6 feet), depending on the dam. Tailwater at Ice Harbor Dam is virtually unaffected throughout the process. During the first drawdown season, tailwater at Little Goose is unaffected.

C.2 Overview of Existing Systems

C.2.1 General

Fish passage facilities on the four Snake River dams are similar in design and function. Each dam has a collection system, a fish ladder (or ladders), and an exit into the upstream forebay. The collection systems consist of entrances along the powerhouse with some near either shore of the dams. The

entrances tie into a channel that carries the fish to the fish ladder. The fish attraction water pumps pump flow through the channels and out the entrances used to attract the fish. Tailwater is pumped at high volume and low head to attract the adult migrating fish. The ladders are weir- and pool-type ladders that accommodate a total elevation change of roughly 30.5 m (100 feet). The ladders are fed by forebay water, and a constant flow of 2.12 cubic meters per second (m^3/s) or 2,124 Liters per second (L/s) (75 cubic feet per second [cfs]) is maintained under normal operating conditions. The upstream exits allow the fish to enter the forebay under most normal operating conditions.

C.2.2 Ice Harbor

The fish handling facilities at Ice Harbor consist of independent north and south shore facilities. The north shore facilities include a fish ladder with a counting station, a small collection system, and a pumped attraction water supply system. The collection system includes two downstream entrances and one side entrance from the spillway basin. In normal operation, one downstream entrance is used and the other two entrances are closed. The auxiliary water is supplied by three electric, 200-horsepower (hp) pumps that produce 7.1 m^3/s (250 cfs) each at 1.2 m (4 feet) total dynamic head (TDH). All three pumps normally are operated.

The south shore facilities are comprised of a fish ladder with a counting station, two south shore entrances, a powerhouse collection system, and a pumped attraction water supply system. The powerhouse collection system includes two downstream entrances and one side entrance from the spillway basin at the north end of the powerhouse, 12 floating orifices, and a common transportation channel. One of the downstream north powerhouse entrances and four of the floating orifices are used during normal operation. Only one south shore entrance is normally used. The attraction water is supplied by eight electric, 250 hp pumps providing 8.5 m^3/s (300 cfs) each at 1.2 m (4 feet) TDH. Six to eight pumps are normally used to provide the required flows. The excess water from the juvenile fish passage facilities (approximately 5.7 m^3/s [200 cfs]) is routed into the fish pump discharge chamber to provide additional attraction flow.

See Figure C1 for a plan view drawing of the Ice Harbor fish handling facilities.

C.2.3 Lower Monumental

The adult fish handling facilities at Lower Monumental Dam are comprised of north and south shore fish ladders and collection systems with a common attraction water supply. The north shore fish ladder connects to two north shore entrances and to the powerhouse collection system. The powerhouse collection system involves a common transportation channel with two downstream entrances and one side entrance from the spillway basin at the south end of the powerhouse, and 10 floating orifices along the downstream face of the powerhouse. The two north shore entrances, two downstream south powerhouse entrances, and four of the floating orifices are used during normal operation.

The south shore fish ladder has two downstream entrances and a side entrance from the spillway basin. The two downstream entrances are used during normal operation. Again, the attraction water supply is provided from one common pumped system. Fish passage, however, is isolated between the north and south ladder systems.

The attraction water is supplied by three turbine-driven pumps located in the powerhouse on the north side of the river. The turbine drives are powered by head water from the forebay through a 48-inch diameter penstock. Attraction water is pumped into a supply conduit that travels under the powerhouse collection channel, distributing water to the powerhouse diffusers and through the spillway to the

diffusers in the south shore collection system. Each turbine-driven pump is capable of producing 23.6 m^3/s (835 cfs) at 1.2 m (4 feet) TDH. Excess water from the juvenile fish bypass system (approximately 5.7-6.8 m^3/s [200-240 cfs]) is added to the auxiliary water supply system for the powerhouse collection system.

See Figure C2 for a plan view drawing of the Lower Monumental fish handling facilities.

C.2.4 Little Goose

The fish handling facilities at Little Goose are comprised of one fish ladder on the south shore, two south shore entrances, a powerhouse collection system, north shore entrances with a transportation channel through the spillway to the powerhouse collection system, and attraction water supply system. The powerhouse collection system is comprised of four floating orifices, two downstream entrances, one side entrance from the spillway basin on the north end of the powerhouse, and a common transportation channel. The four floating orifices, two downstream entrances at the north end of the collection system, and the south shore entrances are normally used. The north shore entrances are comprised of two downstream facing entrances and a side entrance from the spillway basin. The two downstream entrances are used normally.

The attraction water is supplied by three turbine-driven pumps that pump water from the tailrace into the distribution system for the diffusers. The turbine drives are powered by head water from the forebay. Each attraction water pump provides 24.0 m^3/s (850 cfs) at 1.2 m (4 feet) TDH. Additional water (approximately 5.7 m^3/s [200 cfs]) is supplied to the attraction water supply system from the juvenile fish passage facilities primary dewatering structure.

See Figure C3 for a plan view drawing of the Little Goose fish handling facilities.

C.2.5 Lower Granite

The adult fish passage facilities at Lower Granite Dam include one fish ladder on the south shore, two south shore entrances, a powerhouse collection system, north shore entrances with a transportation channel through the spillway to the powerhouse collection system, and an attraction water supply system. The powerhouse collection system is comprised of 10 floating orifices, two downstream entrances, one side entrance from the spillway basin on the north end of the powerhouse, and a common transportation and fish passage channel. Four of the floating orifices, two downstream entrances at the north end of the collection system, and the south shore entrances are normally used. The north shore entrances are made up of two downstream entrances and a side entrance from the spillway basin. The two downstream entrances are used normally.

Three electric pumps that pump water from the tailrace to the diffusers supply the attraction water. Two pumps are normally used to provide the required flows. The pumping system is comprised of one variable speed pump providing 12.7-29.7 m^3/s (450-1,050 cfs) at 1.2 m (4 feet) TDH using 350 to 800 hp and two pumps providing 29.7 m^3/s (1,050 cfs) at 1.2 m (4 feet) TDH using 800 hp each.

See Figure C4 for a plan view drawing of the Lower Granite fish handling facilities.

C.3 Fish Handling Facility Modification Options Considered

This section discusses the possible options considered for temporary fish passage and why they were or were not selected. Anticipated effectiveness of each alternative as evaluated by fish passage experts was the prime criteria for selection. Implementation, construction, logistics, and schedule were also considered. The selection process was based on the assumptions that this was to be a temporary fish

passage solution and that the drawdown would be performed at Lower Granite and Little Goose Dams simultaneously during the first season, followed by Lower Monumental and Ice Harbor Dams. If a four-project simultaneous drawdown were to take place, slight modifications would be required to implement the selected alternative. The trap-and-transport option would have to be implemented at Ice Harbor Dam only. Fish would be trapped at Ice Harbor and transported above Lower Granite Dam. This option would require pre-tagging of Tucannon River and Lyons Ferry fish prior to drawdown. Pre-tagging the fish would allow detection and separation during the trap-and-transport process allowing them to be placed in the Lower Monumental reservoir. The options that were considered in the selection of the preferred options are presented in Table C1.

Table C1. Options Considered in the Selection of the Preferred Options

Option Descriptions	Advantages	Disadvantages
Do nothing. This option requires operating the existing systems as long as criteria can be met during the drawdown process. Passage systems would be shut down when water surface elevations reached levels too low to operate the systems. Fish would pause on their upstream migration until the reservoir was evacuated and the embankments were removed.	• This option would be the easiest to implement, requiring no significant modifications.	• Fish runs would be held up during the drawdown period. This would create adverse conditions that are biologically unacceptable. Fall chinook, for instance, might spawn in areas of the reservoirs that could dry up during drawdown, thereby exposing their redds.
Trap adults at each dam and transport from tailrace to forebay. This option consists of installing a new trapping facility in the existing adult migration system at each dam. As fish are trapped they would be transported above the dam and released back into the forebay. Transportation could be by truck or other type of conveyor.	• This option is desirable because it allows fish to migrate at a similar pace as in the past. Lower Granite Dam is already equipped with an adult trapping facility in the fish ladder, which would minimize modifications required at that dam.	• The attraction water systems at each dam (except Ice Harbor) would have to be modified by adding auxiliary pumping plants to provide adequate water flows at the potentially lower tailwater elevations. In the case of Little Goose, an auxiliary pumping plant is required because the existing attraction water pumps are turbine driven causing them to become inoperable during drawdown of the forebay. • Expensive and complex construction is required. • Adult fish entrances would have to be modified (except at Ice Harbor) by extending them with ladders to the lower tailwater elevations. Modifications at Little Goose would be a little less complicated and be comprised primarily of isolating the north shore entrances. • During the drawdown period, adult fish migration numbers can approach 4,000 fish per day. The system would have to be designed for this maximum amount to avoid "stacking up" fish while they wait for transport. • Most of the existing fish trucks are designed for transporting juveniles and would have to be modified to accept both adults and juveniles. Also, given the limited number of adult fish allowed per truck, the logistics of trucking becomes very difficult. • This requires extensive handling of fish, which is undesirable. Each fish would be trapped and transported twice during each drawdown season.

Table C1. Options Considered in the Selection of the Preferred Options

Option Descriptions	Advantages	Disadvantages
Trap adults at the downstream dam and transport above the next dam upstream in each drawdown season. This option consists of installing a new trapping facility in the existing adult system at Little Goose and Ice Harbor Dams during the first and second seasons of drawdown respectively. As fish are trapped, they would be transported to some point above the next dam upstream and released back into the river system.	• This option is desirable since Little Goose and Ice Harbor Dam's tailwater elevation remains unchanged during their respective drawdown seasons. The adult attraction system would still function up to a point in the fish ladder. Only slight modifications and minimum pumping would be required to install a functioning trap. • One trap-and-haul operation per season minimizes fish handling.	• Most of the existing fish trucks are designed for transporting juveniles and would have to be modified to accept both adults and juveniles. • During each drawdown season adult fish migration numbers can approach 4,000 fish per day. The limited number of fish allowed in one tanker truck makes the logistics of trucking very difficult. • Stray Columbia River fish may get mixed in with the trapped Snake River fish primarily during the second season drawdown while fish are being trapped at Ice Harbor.
Install new and modified fish ladder systems at each dam to maintain upstream migration facilities. Existing ladder systems are designed to elevate fish approximately 30.5 m (100 feet) and would quit functioning shortly after the drawdown has begun. New systems would be designed to adjust to varying pool elevations in both the forebay and tailrace as required, thereby allowing the fish ladders to function through drawdown.	• Typical fish passage would be maintained during the period of changing river elevations and flow paths. • Pumping requirements could be minimized if the overall lift of the ladders was reduced as the forebay elevations approach the tailwater elevations.	• This option would require almost a completely new adult passage system. Entrances and tailrace passages would be the only key existing items that could be used. • Construction would be extensive and costly. • Scheduling this work would be difficult due to long construction times. The installation would have to be completed prior to drawdown, resulting in extensive in-water work that could disrupt existing adult system operations. • Maintaining existing systems requires lifting the fish the original 30.5 m (100 feet), which seems impractical as pool elevations approach each other on either side of the dam.
Use existing temporary fish ladders. A number of various temporary fish ladders were constructed to maintain adult fish passage during construction of the dams. The study team researched existing temporary systems to determine the possibility of recommissioning them for use during drawdown.	• Systems already exist and have proven to be effective to some degree.	• The systems were designed for construction of the dams as opposed to the breaching of existing embankments. The differing sequence of events in the drawdown would not allow making the temporary ladders to be functional prior to achieving dam breach. • Most of the existing systems have been filled in with concrete or removed causing recommissioning to be an extensive and, in some cases, perhaps impossible task.

Table C1. Options Considered in the Selection of the Preferred Options

Option Descriptions	Advantages	Disadvantages
Modify existing systems. This option requires the use of modified fish passage entrances and exits and pumping additional attraction and ladder water to the existing systems to maintain adult fish passage.	• Normal fish handling facility operation is maintained. • Required construction modifications are minor when compared to other options. • Most modifications could be performed without disrupting the existing system operation. • Equipment used during the first drawdown season may be able to be salvaged and reused as applicable in the second season drawdown.	• This option requires extensive in-water and diving work for installation. • Auxiliary pumping systems would be fairly costly to operate. • Entrance and exit conditions are not ideal and may affect fish passage efficiency.
Use existing navigation lock features for fish passage. This option would utilize the navigation lock fill and empty conduits as a temporary method for fish passage.	• The system is already in place and will be taken out of service early in the drawdown process. This permits it to be used for other purposes as feasibility allows.	• Passing flow through this system creates hydraulic conditions not suitable for fish passage without extensive modifications.

C.4 Recommended Options

The migration of adult fish must not be impeded for any extended length of time. Since drawdown activities will render existing fish passage facilities inoperable, alternate means must be provided during this period.

The range of options considered for this alternate means was narrowed to two options by the study team: 1) modify fish entrance, exit, and fish ladders so that fish passage may continue at each project during each drawdown season, and 2) construct a fish trap at both Little Goose and Ice Harbor Dams, collect adults, and truck-transport anesthetized adults to an appropriate discharge point above the next upstream dam during each drawdown season.

The river conditions during drawdown are expected to be very difficult for adult fish migration. Specifically, the concentration of sediment in the water may create both passage problems and health problems for migrating adult fish. This is the primary reason that the option to trap the adults and truck-haul them to a discharge location above the areas most affected is the preferred option. However, other drawdown configurations include drafting all four reservoirs concurrently. In this case the trap and truck-haul option implemented would be to trap adults at Ice Harbor Dam and truck-haul them to a point above Lower Granite Dam. One problem posed with this option is that Tucannon River and Lyons Ferry fish would be encountered. These fish need to be placed in the Lower Monumental reservoir creating further complications with this option. The largest complication is identifying and separating these fish in the Ice Harbor adult trap. Therefore, in a four reservoir concurrent drawdown, modifying existing systems may prove to be the preferred option. Consequently, the study team examined both options in this study.

C.4.1 Option 1 - Modify Each Existing System

System Changes

Because the modifications planned for this option are temporary measures, the study team believed it would be prudent to operate only the most efficient collection system at each dam. Currently, the ladder and attraction systems on the powerhouse side of the dams collect and pass approximately 85 percent of the fish when the dams are not spilling. The shore entrances on this side near the beginning of the ladder currently handle most of the adult fish. These systems should be maintained and operated throughout the drawdown and channelization process. This would result in the following system changes at each dam during their respective drawdown seasons:

- System changes at Ice Harbor Dam would involve shutting down the north shore ladder and attraction system, and operating only the south shore ladder and attraction system.
- System changes at Lower Monumental Dam would involve shutting down the south shore ladder; installing bulkheads to stop water flow through the fish transportation and attraction water conduit to the powerhouse and south shore system; and operating the north shore entrances only.
- System changes at Little Goose Dam would involve installing bulkheads to stop water flow through the fish transportation and attraction water conduit to the north shore system and powerhouse, and operating the south shore entrances only.
- System changes at Lower Granite Dam involve installing bulkheads to stop water flow through the fish transportation and attraction water conduit to the north shore system and powerhouse, and operating the south shore entrances only.

There is no historical data to say how overall efficiency would be affected by operating the powerhouse system only. Operating just one ladder/attraction system maintains adult fish passage while greatly

reducing cost and construction required at both Ice Harbor and Lower Monumental dams. It also reduces costs to operate the systems on all four projects during drawdown. By operating the powerhouse systems only, savings can be realized because of fewer entrance modifications, exit modifications, and reduced pumping costs, without greatly affecting passage efficiency. See Figure C10 for a typical overall plan of recommended system modifications.

Entrance Modifications

As previously mentioned, tailwater elevation changes during this process would be a function of river flow and would not be drastic compared to forebay changes. Existing dam tailwater elevations are near natural river flow elevations in the areas of the dams during certain river flow conditions. This makes it feasible to extend the existing fishway entrances out and down to connect with varying tailwater elevations where required.

Existing powerhouse fishway entrances normally used near the beginning of the fish ladder would be the ones operated during drawdown. Existing entrances would be "extended" to reach lowering tailwater elevations using specially designed ladders connected to the entrances. To manage changing tailwater elevations, the ladders would pivot at the connection to the existing entrance with the inlet end of the ladder being buoyant, allowing them to fluctuate with varying tailwater elevations. Two ladder sets would be used at dams having possible tailwater elevation drops greater than 3.0 m (10 feet) below normal minimum operating elevation. One ladder would be designed for the first half of drawdown, and the other set for the final stages of drawdown. This would keep ladders within the acceptable range of slopes for proper operation. The ladders would be designed to allow a quick change of ladder sets as water level drops below the range of the first ladder set. A schematic representation of a new ladder entrance is shown in Figure C5. Existing adjustable entrances would be dogged off at elevations required for operating the collection systems at minimum flows and depths. This would eliminate any possible variables in operation and minimize the cost of pump installation and operation.

A Denil-type ladder is currently planned for the entrance extensions. The actual type of ladder to be implemented may or may not be a Denil and will require further research and model testing. The Denil ladder is a fishway that consists of a rectangular chute with closely spaced baffles or vanes located along the sides and bottom, as shown in Figure C6. Velocity is varied within the chute by geometry; low at the bottom and higher at the top. Flows within the fishway would be designed to provide adequate attraction velocities.

Based on this overall approach, the following entrance modifications would be made at each of the dams:

- No entrance modifications would be required at Ice Harbor Dam, and tailwater would be unchanged.
- The two north shore entrances at Lower Monumental Dam would be fitted with new Denil ladder entrance extensions. The minimum tailwater possible is about 2.9 m (9.5 feet) below present minimum operating tailwater. Only one set of ladders would be necessary.
- No entrance modifications would be required at Little Goose Dam. During the first drawdown season the tailwater at Little Goose remains unchanged.
- The two south shore entrances at Lower Granite would be fitted with new Denil ladder entrance extensions. Minimum tailwater possible is about 4.6 m (15.2 feet) below the current minimum operating tailwater. Two complete sets of ladders would be supplied: one set designed for the first 2.4 m (8-foot) drop in tailwater, and the other set designed for the remainder.

Attraction Water Modifications

Fishway flows and depths would be operated at current minimum operating conditions. This would allow the most economical functioning of the system. Collection channel velocities must be maintained at 0.6-to-1.2 meters per second (mps) (2-to-4 feet per second [fps]) and fishway depths must be kept at a minimum 1.8 m (6 feet).

Existing attraction water pumps are not designed to operate at heads greater than about 1.2 m (4 feet), even if retrofitted with larger drive motors. As the tailwater elevation drops, the existing attraction water pumps at Lower Granite and Lower Monumental will fall below their operating ranges during the first and second season drawdowns, respectively. During this time the pumps cannot provide minimum required flows into the collection systems. In the present condition, collection channel elevation fluctuates up and down with tailwater elevation maintaining constant head across the attraction water pumps. During drawdown, the head across the attraction water pumps at Lower Granite and Lower Monumental would increase due to a lowering tailwater elevation with constant collection channel elevation. Existing attraction water pump intakes would remain submerged.

The additional head on the system creates a problem. Changing out the attraction water pumps is not feasible because the systems must remain intact up to the point of drawdown. It would be difficult and very costly to install a new pumping system capable of operating in both modes, normal and throughout drawdown. Also, although tailwater remains unchanged at Little Goose Dam during the first drawdown season, the attraction water pumps are turbine driven by forebay head so they will become inoperable as the drawdown begins.

Consequently, the study team recommends adding new pumps to the current systems and tying them into the existing attraction water supply conduits, as shown in Figure C7. The new pumps would provide flows and heads similar to the present systems, allowing them to operate as they currently do. It is possible that these pump systems might be rented, thereby reducing the capital cost. Also, it may be feasible to salvage systems used during the first drawdown season and use them in the second drawdown season. Specifically, the study team proposes the following attraction water modifications at each of the dams:

- The existing attraction water pump system at Ice Harbor would operate as normal. However, there would be no auxiliary flow from the juvenile dewatering system.
- The total attraction water required at Lower Monumental Dam would be 14.2 m^3/s (500 cfs), which is 7.1 m^3/s (250 cfs) for each entrance. Flow numbers would exceed present range of operation for these entrances. Maximum total head on the system, including losses, would be about 4.11 m (13.5 feet) at minimum river flow. Two 600-hp axial flow pumps would be installed, each producing 7.1 m^3/s (250 cfs) of attraction water flow.
- Total attraction water required at Little Goose Dam would be 28.2 m^3/s (1,000 cfs) to operate the powerhouse and south shore collection systems as they are presently operated. Head on the system would remain unchanged during the first drawdown season. Three 300-hp axial flow pumps would be installed, each producing 9.4 m^3/s (333 cfs) of attraction water flow.
- Total attraction water required at Lower Granite Dam would be 14.2 m^3/s (500 cfs), which is 250 cfs for each entrance. Flow numbers would exceed the current range of operation for these entrances. Maximum total head on the system, including losses, would be about 5.85 m (19.2 feet) at minimum river flow. Three 600-hp axial flow pumps would be installed, each producing 4.7 m^3/s (167 cfs) of attraction water flow.

Fish Ladder Flow

Fish exit and auxiliary fish ladder flow is typically provided by gravity-fed forebay head water. Fish ladder flows, both auxiliary supply and fish exit flows, will drop off and discontinue shortly after the drawdown has begun. Ladder flows must be maintained at current minimum levels to provide adequate depths and velocities for fish passage.

Where applicable, existing forebay intakes for the ladder flows would be closed or isolated.

Water supply for both the fishway exit and auxiliary supply to the ladder would be provided by adding a series of pumps in the forebay and plumbing them in together to supply these features. These pumps might also be rented in order to reduce overall cost of this modification. Also, it may be feasible to salvage systems used during the first drawdown season and use them in the second drawdown season. A number of pumps would be provided so they can be "staged" as the reservoir drops, providing the required constant fish ladder flow at varying forebay elevations. Also, a throttling valve will be placed in the system to create required head loss keeping the pumps operating on the proper areas of the pump curves as water surface elevations vary. The pumps would be mounted on a platform on the upstream face of the dam, as shown in Figure C8.

The specific fish ladder flow modifications required at each dam are as follows:

- At Ice Harbor Dam, 2.7 m^3/s (96 cfs) would be provided to the false weir and auxiliary fish ladder flow system. Present minimum forebay elevation is 133 m (437 feet). Drawdown elevation would be approximately 103 m (338 feet). Four vertical turbine pumps capable of producing 0.68 m^3/s (24 cfs) each at a TDH of about 34 m (110 feet) would be required. Each pump would be driven by a 450 hp electric motor.
- The Lower Monumental north shore fish ladder would have 2.1 m^3/s (75 cfs) provided to the false weir and auxiliary ladder flow system. Current minimum forebay elevation is 164 m (537 feet). Drawdown elevation will be approximately 130 m (427.5 feet) at minimum river flow. Four vertical turbine pumps capable of producing 0.53 m^3/s (18.75 cfs) each at a TDH of about 37 m (120 feet) would be required. Each pump would be driven by a 400 hp electric motor.
- The fish ladder at Little Goose would have 2.1 m^3/s (75 cfs) provided to the false weir and auxiliary ladder flow system. Current minimum forebay elevation is 193 m (633.0 feet). Drawdown elevation would be approximately 158 m (517.4 feet) at minimum river flow. Four vertical turbine pumps capable of producing 0.53 m^3/s (18.75 cfs) each at a TDH of about 38 m (125 feet) would be required. Each pump would be driven by a 400 hp electric motor.
- The fish ladder at Lower Granite would have 2.1 m^3/s (75 cfs) provided to the false weir and auxiliary ladder flow system. The current minimum forebay elevation is 223 m (733.0 feet). Drawdown elevation will be approximately 188 m (617.8 feet) at minimum river flow. Four vertical turbine pumps capable of producing 0.53 m^3/s (18.75 cfs) each at a TDH of about 38 m (125 feet) would be required. Each pump would be driven by a 400 hp electric motor.

Exit Modifications

Since the forebay water surface elevations will fall below the fishway exits soon after drawdown begins, a system to safely transport the fish from the top of the ladder into the lowering forebay will need to be provided.

A system similar to the alternate fish ladder exit that currently exists at Lower Granite Dam would be retrofitted to the fish handling systems. This system consists of a false weir and a release flume system that will work in conjunction with the pumped ladder flow mentioned earlier. The pumped ladder flow

supplies the false weir with attraction water to help attract the fish into the exit flume and also provides flow down the ladder and flow down the exit flume to help flush the adult fish.

The false weir would be fit to the existing fish ladder exit with a transition into a 450-mm (18-inch) wide by 700-mm (27.5-inch) tall round bottom fiberglass flume system dropping at a 20 percent slope. The flume system would be located near the exits and extend down in a series of straight lengths and long radius bends so it can be supported off the face of the dam. The flume would have an open top allowing the transported fish to exit the system at any water surface elevation, as shown in Figure C9. The false weir and exit flume system would be installed at all four dams.

Schedule

Current systems would be operated in their normal mode as long as possible through the beginning stages of each drawdown season. Ice Harbor and Lower Monumental Dams would operate as they normally do during the first drawdown season. Lower Granite and Little Goose Dams would be breached and have free flowing river during the second drawdown season and adult systems would not be required. Affected adult fish exits would become inoperable within the first couple of days of drawdown. Attraction water systems and fishway entrances, however, may operate for quite some time throughout the drawdown process in their normal modes, except for the turbine-driven attraction water pumps at Lower Monumental and Little Goose Dams. River flow at the time of the drawdown would govern when the new temporary systems will have to be employed.

New ladders would be fabricated prior to drawdown. All pivoting and flexible seal connections to the existing structure would have to be performed in the wet using dive crews prior to drawdown. When tailwater elevation falls below that required for normal entrance operation, the portion of system not to be used would be bulkheaded off, and new ladder entrances attached.

New attraction water pumping systems would have to be installed and electrically connected prior to beginning river drawdown. Pump supports, pumps, shafting, valving, and taps into the attraction water supply conduit would have to be done in the wet using dive crews. Taps into existing attraction water supply conduits would have to be performed during a normal adult passage system outage the year prior to drawdown. Motor supports, drive motors, and electrical connections would be assembled and installed on the tailrace deck.

The attraction water conduits and fish passage conduits would have to be modified for isolation bulkheads during a normal system outage a year prior to drawdown.

New exit flume systems and an upwell and false weir caisson connection to existing fishway exit would have to be installed in the wet prior to beginning drawdown using dive crews. The upwell and false weir caisson would have to be designed for "drop-in" type connection to existing fishway exits, to new exit flume system, and to ladder water supply piping providing a quick installation of the system as drawdown progresses past the point of normal operation.

Fish ladder flow pumping stations would have to be installed and functional prior to beginning drawdown. Pumps, discharge piping, header pipe, throttling valve and pipe and pump column supports would have to be installed in the wet using dive crews. Motors and electrical connections would be installed on the top decks.

C.4.2 Option 2 - Trap Adults at the Downstream Dam and Transport Above the Next Dam Upstream in each Drawdown Season

General

This option involves trapping all migrating adult fish at Little Goose Dam and transporting them above Lower Granite Dam for release during the first drawdown season. During the second drawdown season, migrating adult fish would be trapped at Ice Harbor Dam and transported above Lower Monumental Dam and released back into the river system.

Enough fish trucks and personnel would have to be available to handle the peak load of migrating fish, approximately 3,000 fish per day. Assuming 50 fish per truck, a 6-hour turnaround time for transport and release, and round-the-clock operation, a minimum of 13 trucks would be required. It may be possible to borrow fish hauling trucks from other agencies or quickly retrofit used tankers for fish hauling purposes. This study team assumed that there would be 10 existing trucks available and 3 that must be bought and modified to suit fish hauling. All new and modified trucks would be designed to haul both juvenile and adult fish so that the tankers could be better utilized.

Required Modifications to Existing Systems

To accomplish this option, the following modifications would need to be made to Little Goose Dam for the first drawdown season:

- The north shore fish attraction system would be isolated as in Option 1. The south shore attraction and fishway entrance system would be operated as it currently is. Tailwater elevations would remain the same.
- Since the attraction water pumps are turbine driven, they would become inoperable as the forebay drawdown began. Three 300-hp axial flow pumps would be installed to supply the required attraction water similar to Option 1.
- The first bend in the fish ladder would be fitted with a diversion screen. The diversion screen would be designed to deploy into place when the trap is ready to operate. Slots would be cut into the ladder walls in this area for the placement of a false weir for fish attraction, similar to that shown in the plan view in Figure C12.
- To operate the new trap and supply ladder attraction water, 2,406 L/s (85 cfs) of pumped flow will have to be provided. As the drawdown begins flow from the top end of the ladder would cease. The current minimum forebay elevation is 193 m (633 feet). The drawdown elevation would be approximately 158 m (517.4 feet). About 1,841 L/s (65 cfs) would be supplied into the top of the fish ladder, with 566 L/s (20 cfs) being piped down to the new fish trap. Approximately 283 L/s (10 cfs) of the fish trap supply will enter the fish ladder through the new false weir at the trap, providing a total ladder flow in the lower reaches of 2,124 L/s (75 cfs) as currently required. The pumping system would be similar to Option 1 for fish ladder flow. In this case, four vertical turbine pumps capable of producing 602 L/s (21.25 cfs) each at a TDH of about 36.6 m (120 feet) would be required. Each pump would be driven by a 450-hp electric motor. Figure C8 shows a similar arrangement to accomplish the auxiliary flow.

To accomplish this option, the following modifications would need to be made to Ice Harbor Dam for the second drawdown season:

- The north shore fish ladder would be taken out of service. The south shore attraction and fishway entrance system would be operated as it currently is. Tailwater elevations would remain the same.
- The return bend at floor elevation 121 m (397.0 feet) of Section No. 1 of the fish ladder would be fitted with a diversion screen. The diversion screen would be designed to deploy into place when the

trap is ready to operate. Slots would be cut into the ladder walls in this area for the placement of a false weir for fish attraction, as shown in the plan view in Figure C12.

- To operate the new trap and supply ladder attraction water, 3,002 L/s (106 cfs) of pumped flow will have to be provided. As the drawdown begins, the fish ladder water supply diffusers operated by the attraction water pumps would remain in service, but the flow from the top end of the ladder would cease. The current minimum forebay elevation is 133 m (437 feet). The drawdown elevation would be approximately 103 m (338 feet). About 2,435 L/s (86 cfs) would be supplied into the top of the fish ladder, with 566 L/s (20 cfs) being piped down to the new fish trap. Approximately 283 L/s (10 cfs) of the fish trap supply will enter the fish ladder through the new false weir at the trap, providing a total ladder flow in the lower reaches of 2,718 L/s (96 cfs) as currently required. The pumping system would be similar to Option 1 for fish ladder flow. In this case, five vertical turbine pumps capable of producing 600 L/s (21.2 cfs) each at a TDH of about 33.5 m (110 feet) would be required. Each pump would be driven by a 400-hp electric motor. Figure C8 shows a similar arrangement to accomplish the auxiliary flow.

New Trap System Features

The new trap systems would have a flume starting in the fish ladder wall at a bend in the ladder system with a false weir to attract the adults into the system. The flume would have a PIT tag detector to allow for any desired monitoring that the biologists may want to perform. A switch gate to switch between two holding tanks would be installed. Two holding tanks allows continuation of the trapping function as a fish hauling truck is being loaded.

The individual holding tanks would be fitted with vertical crowders for quick release of the fish. Tanks would be arranged so that the crowders can be operated by a single, strategically located, mobile crane. This would eliminate excessive mechanical equipment, saving costs and maintenance.

Water supply for a variety of purposes, including the false weirs, flume flushing, holding tanks, fish release auxiliary water, and truck loading, will be piped down from the pumped fish ladder water supply system previously mentioned. Flow required is 85 L/s (3 cfs) per holding tank, 28 L/s (1 cfs) per flume flushing point, 28 L/s (1 cfs) per truck loading chute, and 283 L/s (10 cfs) per false weir, with the truck loading water assumed to be incidental. This is a total water supply per system of 566 L/s (20 cfs). About 283 L/s (10 cfs) of the total would enter the ladder system through the false weir and would supplement the total ladder attraction flow required. Tank drains would be discharged into the lower regions of the ladders but really cannot be counted on as ladder flow due to the distance down the ladder that they must discharge into. Therefore, 283 L/s (10 cfs) of addition pumped water needs to be supplied beyond the total ladder flow.

See Figures 11 and 12 for arrangement of the new adult trap features.

Schedules

The false weirs and the diversion screen would have to be installed during the fish ladder outage the year prior to the drawdown season. The diversion screen would be designed to not obstruct normal ladder operation when not deployed.

The water supply pumping station would have to be installed and functional prior to beginning drawdown. Pumps, discharge piping, header pipe, throttling valve, and pipe and pump column supports would have to be installed in the wet using dive crews. Motors and electrical connections would be installed on the top decks. Piping down to the new trap could be installed at anytime prior to drawdown.

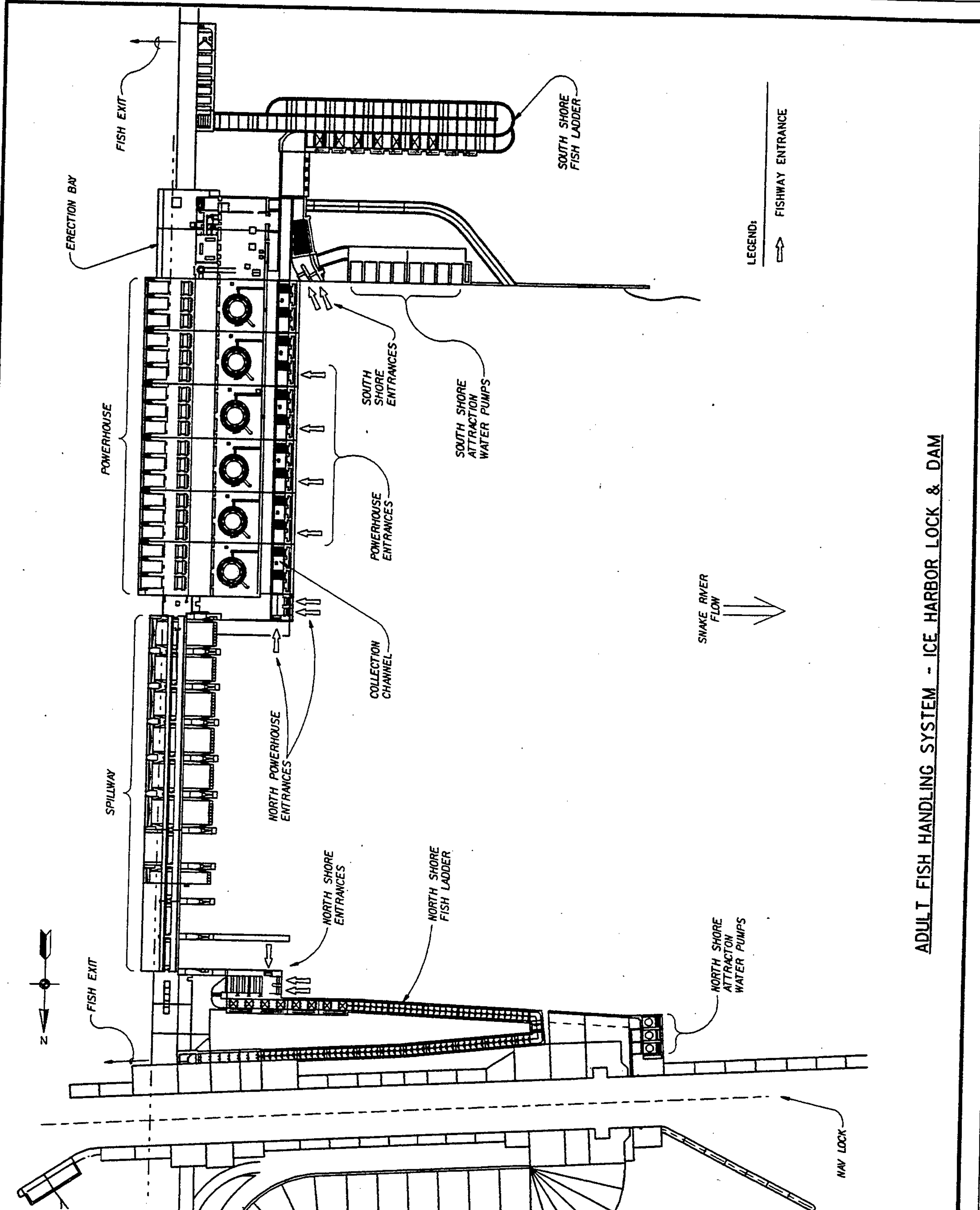
Everything downstream of the new false weir could be constructed at any time prior to beginning drawdown.

Electrical power for new attraction water, fish ladder pumping systems, and adult trap supply would have to be back fed from the nearby power substations. Actual switchgear and circuitry requirements for the pump stations have not been considered in this report and will require detail description and design in any subsequent work.

C.5 Conclusion

The relative advantages and disadvantages of these temporary fish passage measures were discussed at the Fish Facilities Design Review Work Group. The primary concern was with implementing modifications to each project to allow the fish to migrate upriver. The water quality of the river during the initial drawdown may be heavily silt laden and may severely impede the migration of fish. The group was unanimous in recommended that trapping and hauling adult fish, while not a good option, was a better option.

The study team selected the trap and haul option as the preferred option for a temporary means to facilitate the migration of adult fish. This is based on the uncertainty of sediment loads in the river and the difficulty in achieving effective fish ladder modifications.



SHEET MAIN SCALE

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT FISH HANDLING SYSTEM
ICE HARBOR LOCK & DAM

Figure:
C1

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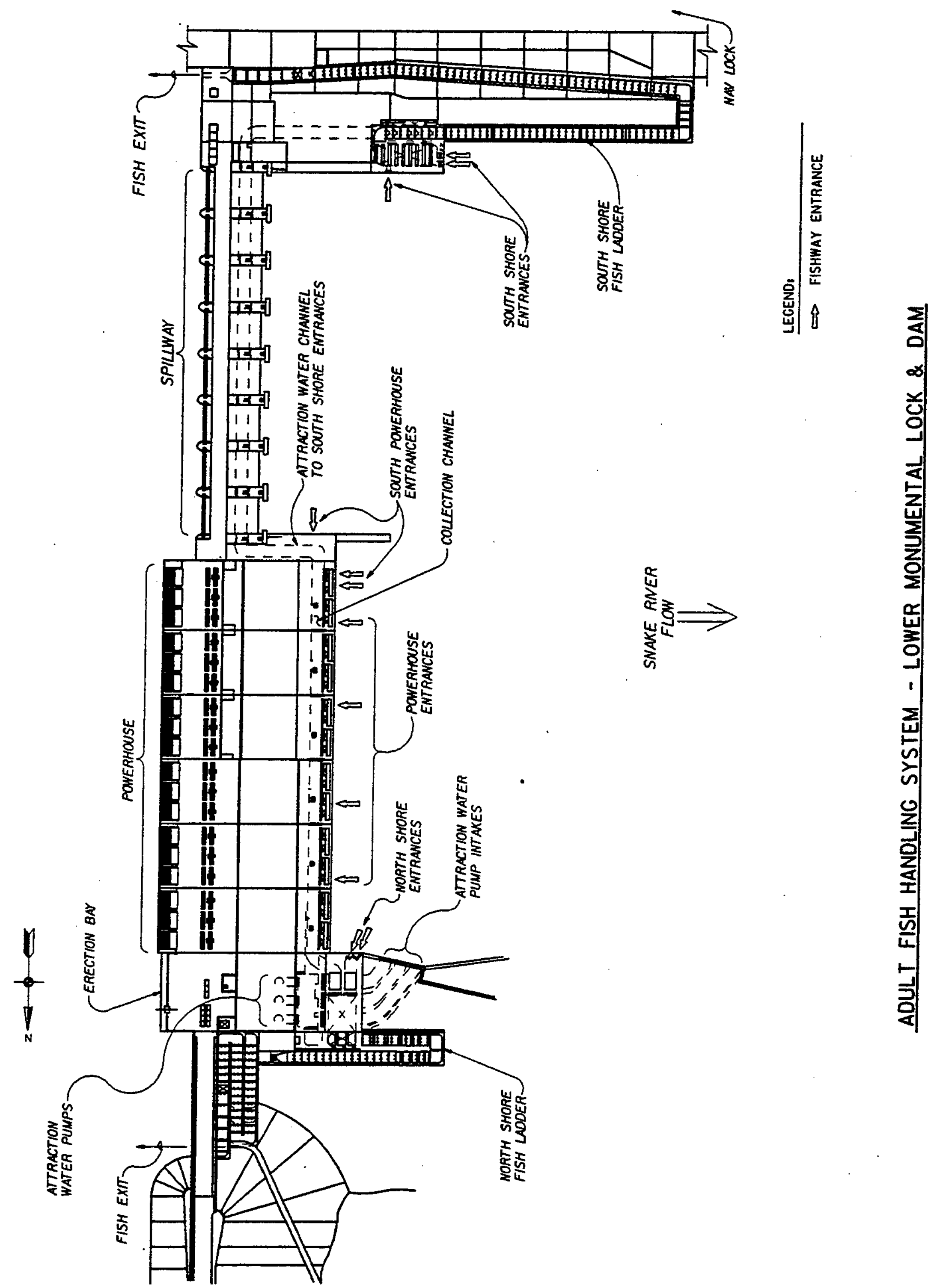

ADULT FISH HANDLING SYSTEM - LOWER MONUMENTAL LOCK & DAM

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT FISH HANDLING SYSTEM
LOWER MONUMENTAL LOCK & DAM

Figure:
C2

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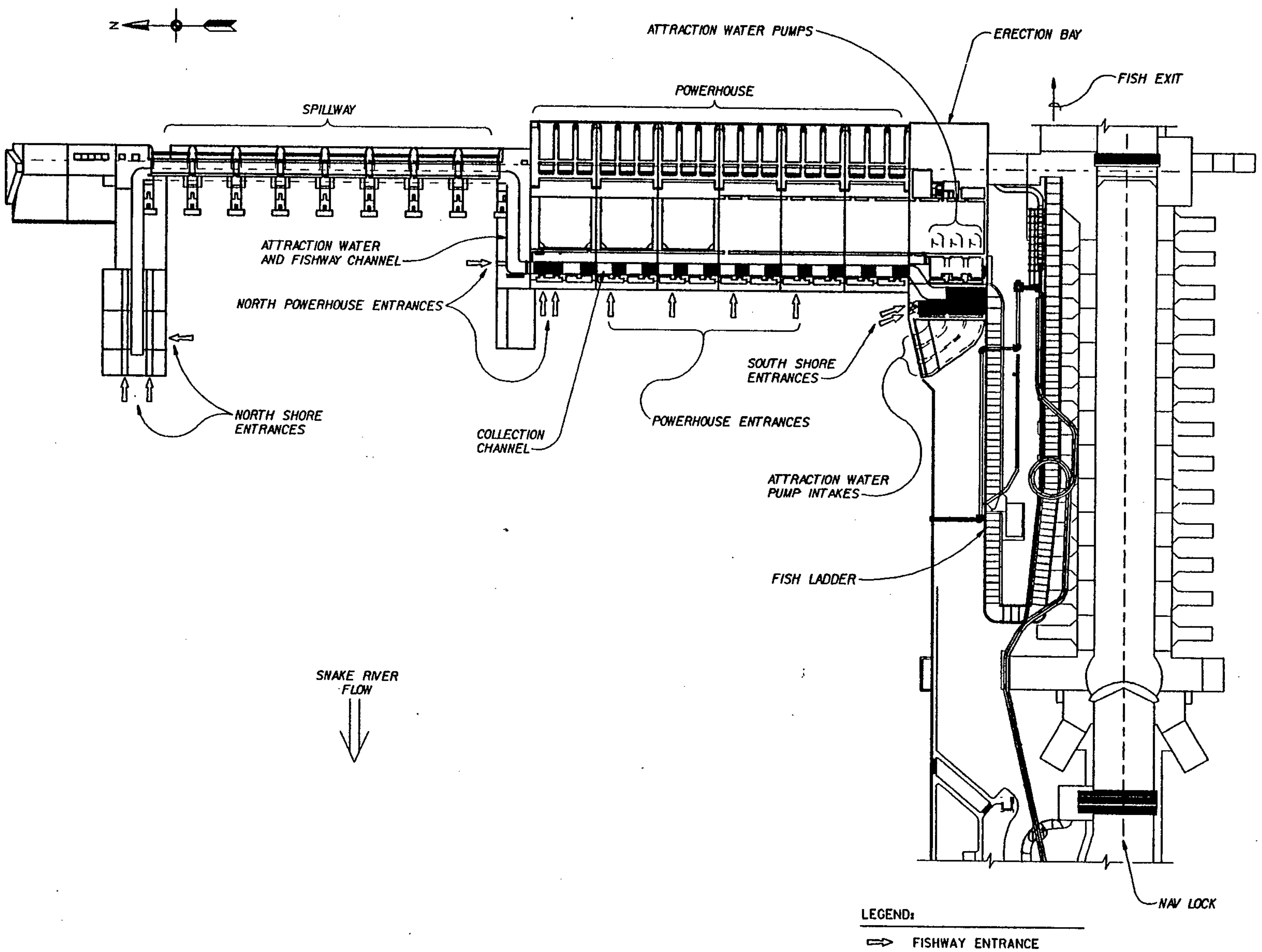

ADULT FISH HANDLING SYSTEM - LITTLE GOOSE LOCK & DAM

US Army Corps
of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT FISH HANDLING SYSTEM
LITTLE GOOSE LOCK & DAM

Figure:
C3

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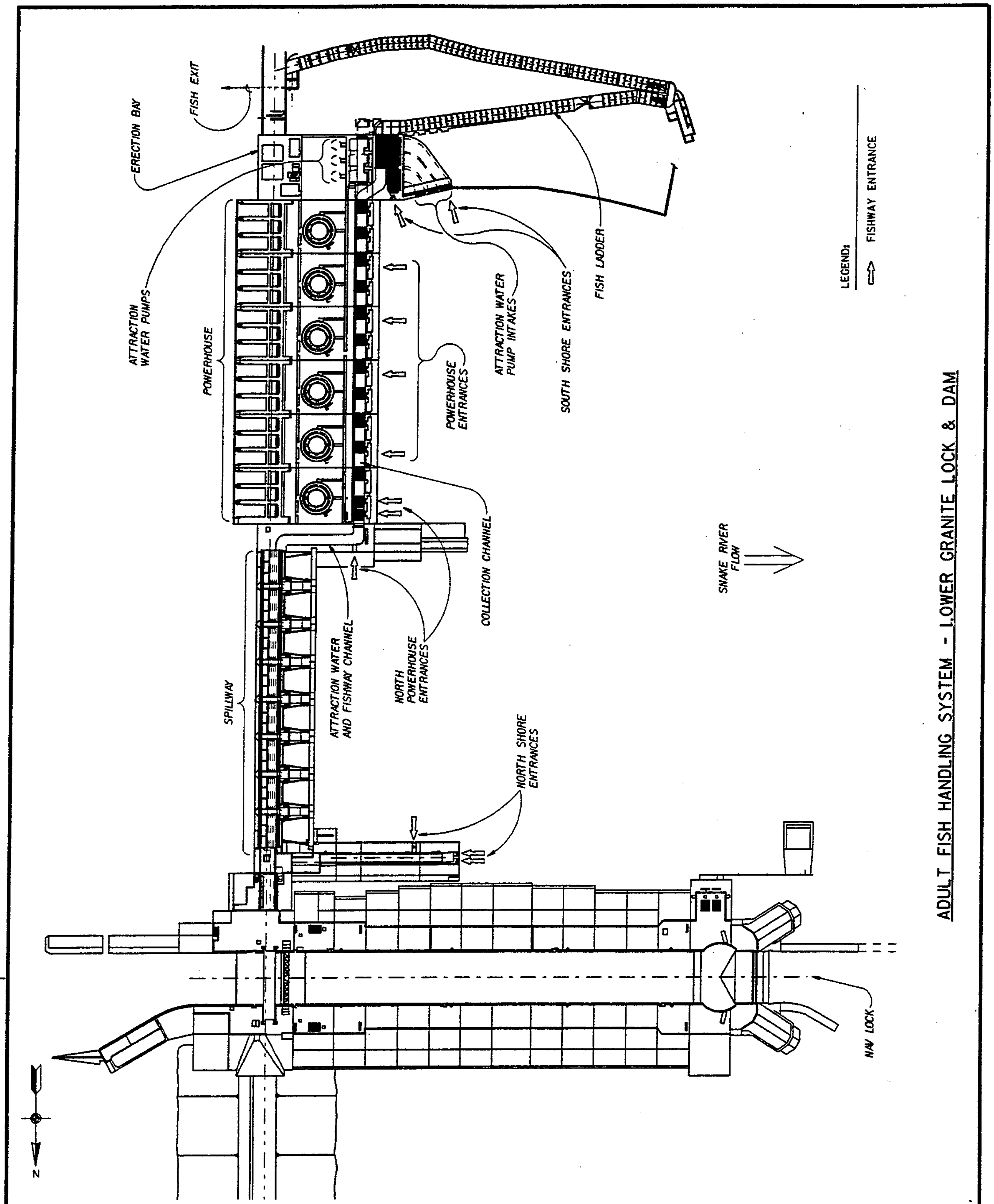

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT FISH HANDLING SYSTEM
LOWER GRANITE LOCK & DAM

Figure:
C4

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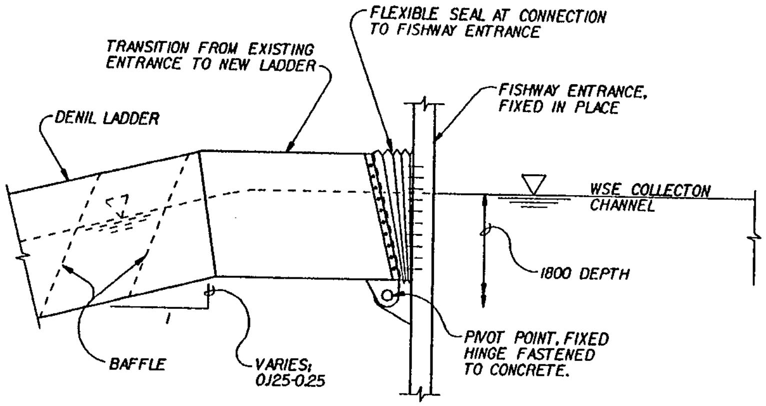
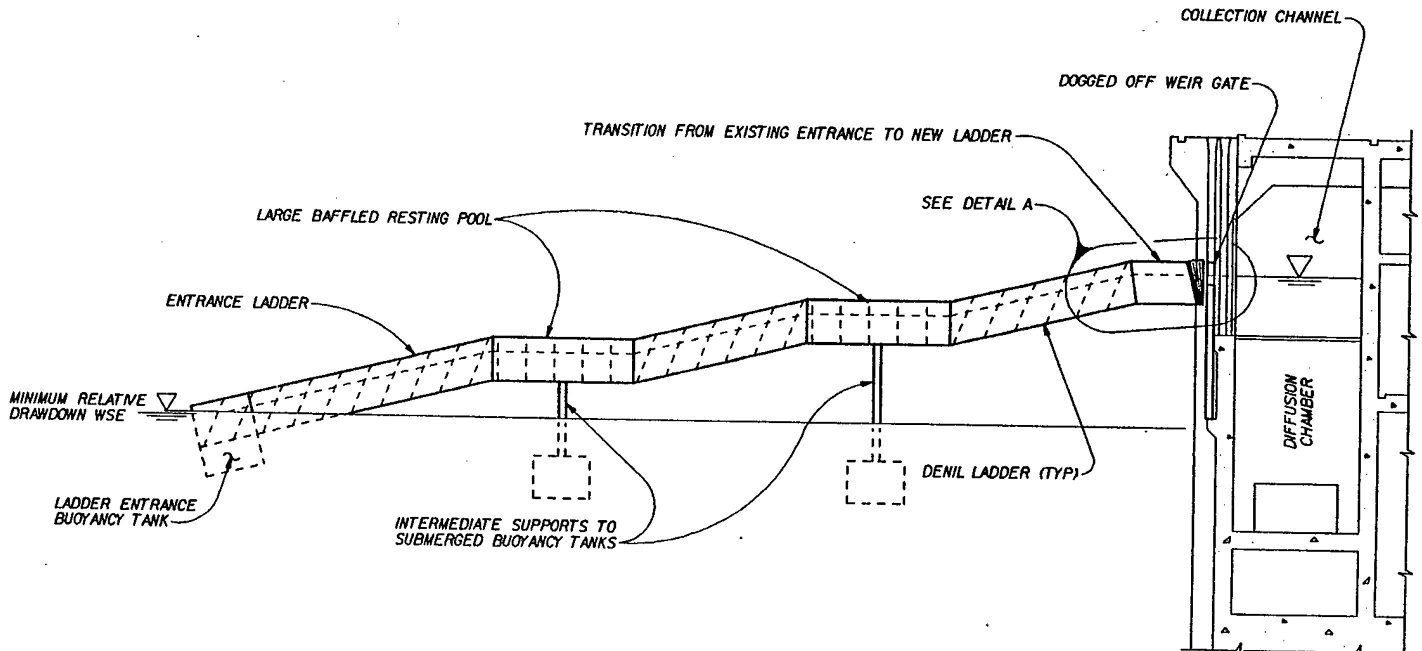


DETAIL A



SECTION AT COLLECTION/ENTRANCE SHOWING LADDER ENTRANCE EXTENSION

NOTE:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN MILLIMETERS.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

SECTION AT COLLECTION / ENTRANCE
SHOWING LADDER ENTRANCE EXTENSION

Figure: C5

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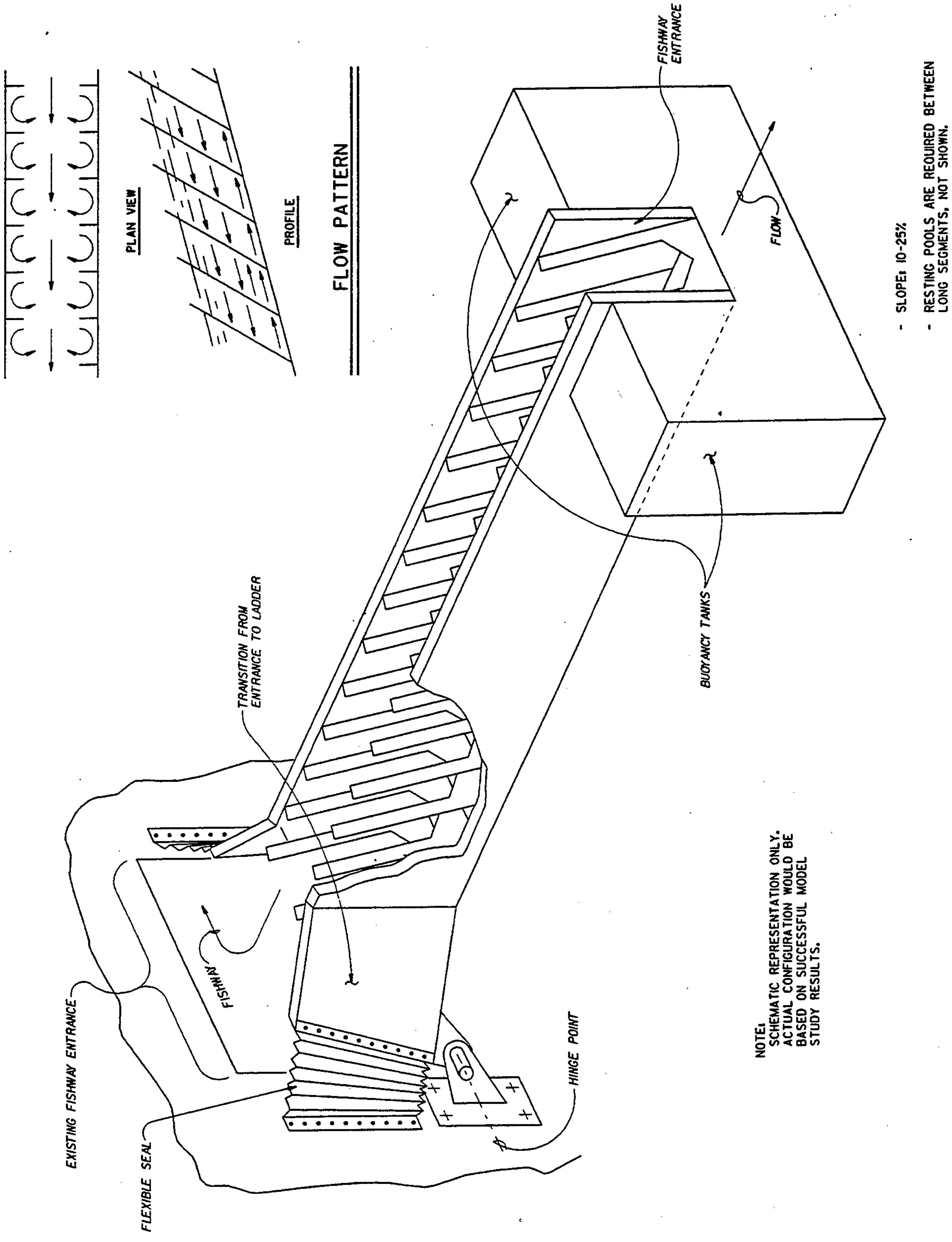

PLAIN DENIL FISHWAY

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

DENIL FISH LADDER
PLAIN DENIL FISHWAY

Figure: C6

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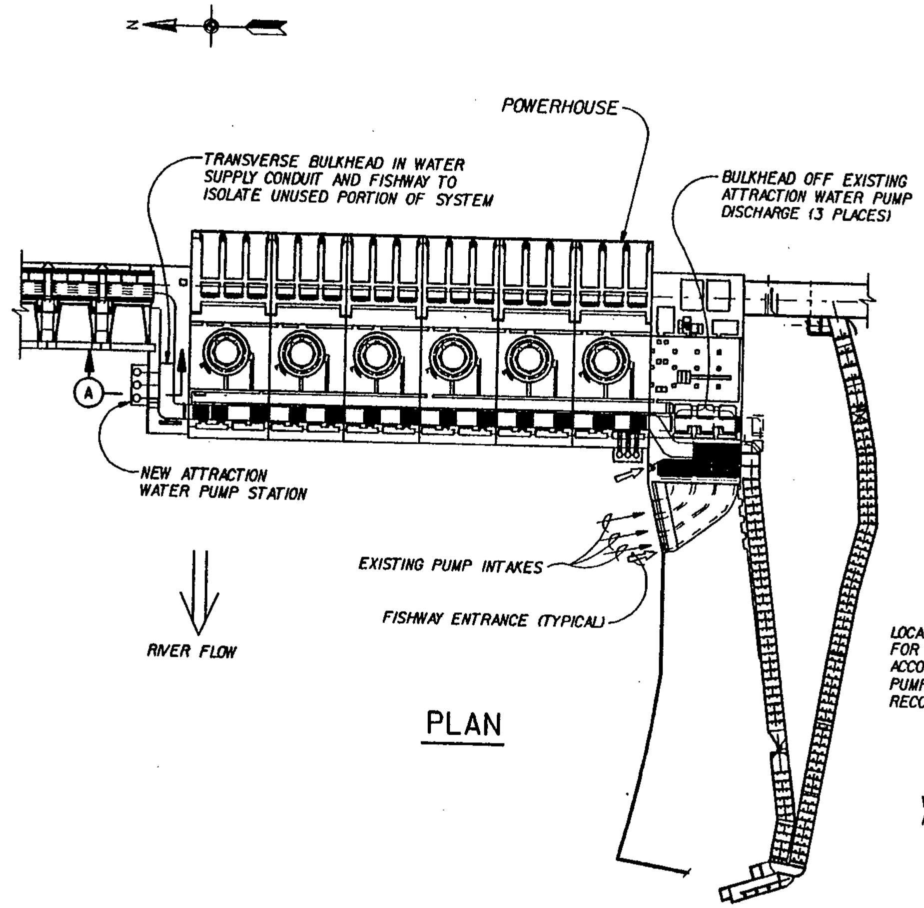
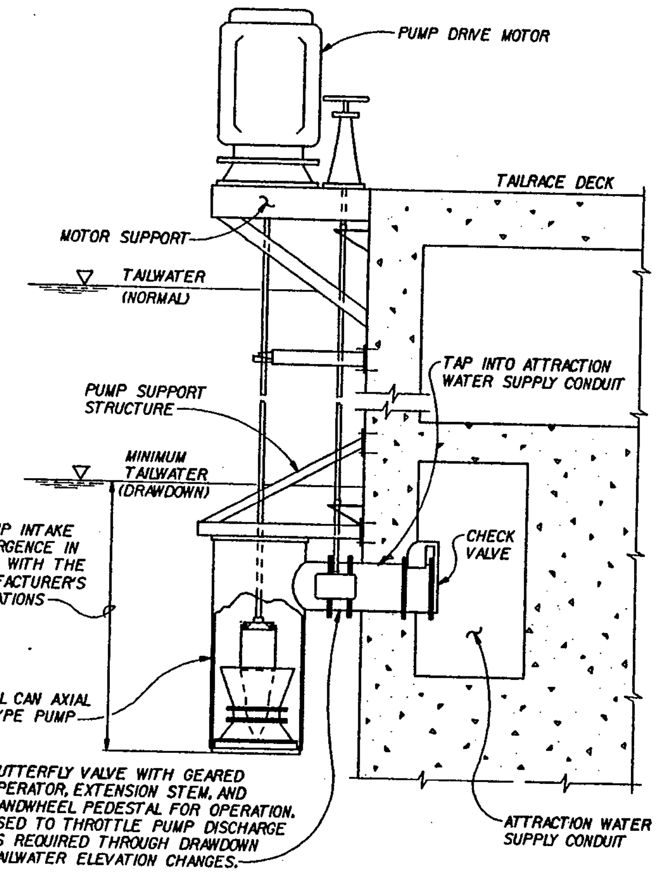


PLAN



SECTION A

Pump Capacity Chart

PROJECT	NO. PUMPS REQUIRED	CAPACITY EACH (CMS)	TOTAL CAPACITY (CMS)	MOTOR (HP) *
ICE HARBOR	N/A	N/A	N/A	N/A
L. MONUMENTAL	2	7.1	14.2	600
L. GOOSE	3	9.4	28.2	300
L. GRANITE	3	4.7	14.2	600

* HP PER MOTOR

EXISTING PUMPS AT ICE HARBOR WILL CONTINUE TO FUNCTION.

NORMAL TAILWATER DURING FIRST DRAWDOWN SEASON. EXISTING TURBINE DRIVEN PUMPS BECOME INOPERATIVE HOWEVER DURING FOREBAY DRAWDOWN. POWERHOUSE AND SOUTH SHORE ENTRANCES TO OPERATE AS ORIGINALLY DESIGNED. ISOLATE NORTH SHORE.

NOTE:
1. CMS - CUBIC METERS PER SECOND

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ATTRACTION WATER PUMPS

Figure: C7

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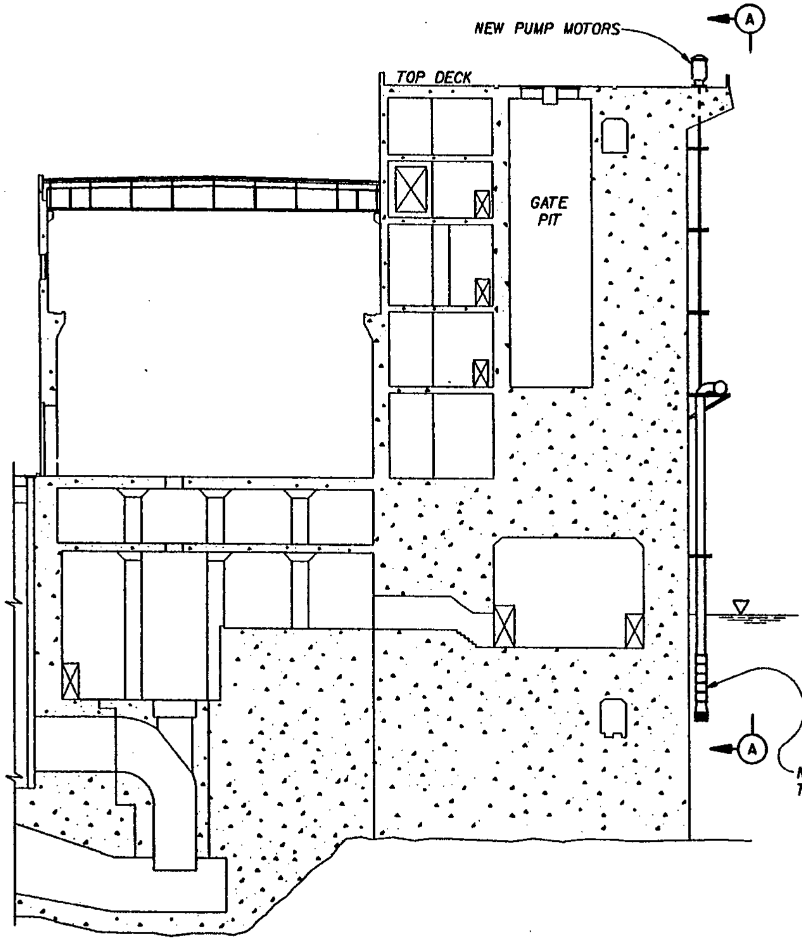
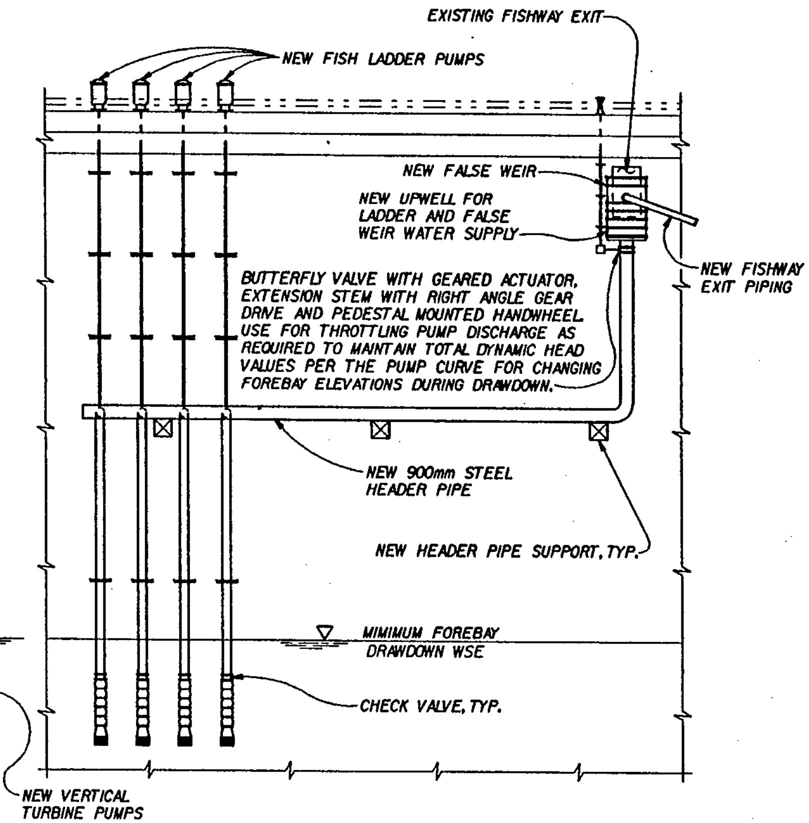
PLOT TIME: 15-OCT-1999 08:13

SHEET MAIN SCALE:



SECTION THRU ERECTION BAY



SECTION A

Pump Capacity Chart

PROJECT	CAPACITY EACH (CMS)	TOTAL CAPACITY (CMS)	MOTOR (HP)
ICE HARBOR	0.68	2.7	450
L. MONUMENTAL	0.53	2.1	400
L. GOOSE	0.53	2.1	400
L. GRANITE	0.53	2.1	400

NOTE:
1. CMS - CUBIC METERS PER SECOND

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

FISH LADDER PUMPS

Figure: C8

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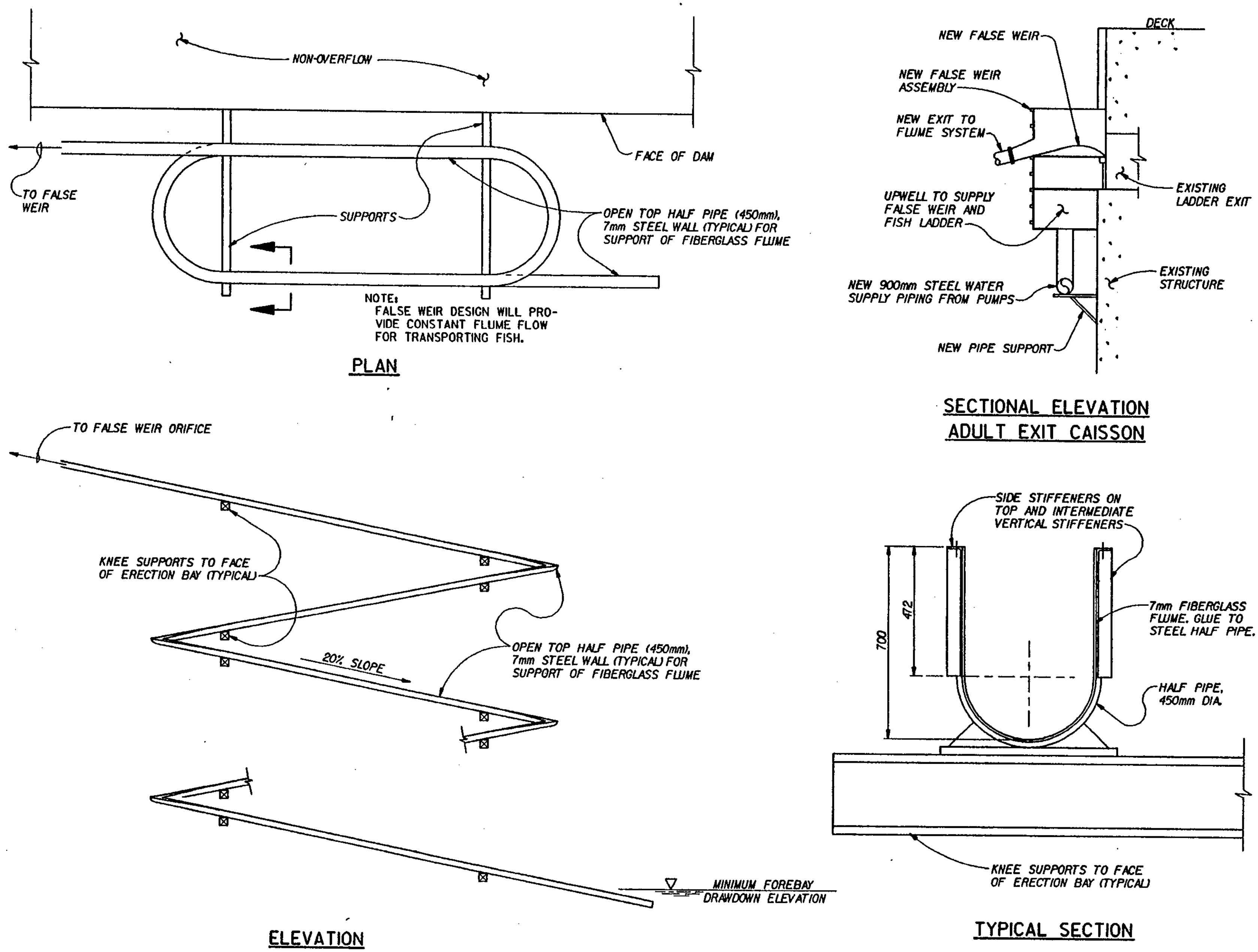

NOTE:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN MILLIMETERS.

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Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LADDER EXIT MODIFICATIONS

Figure: C9

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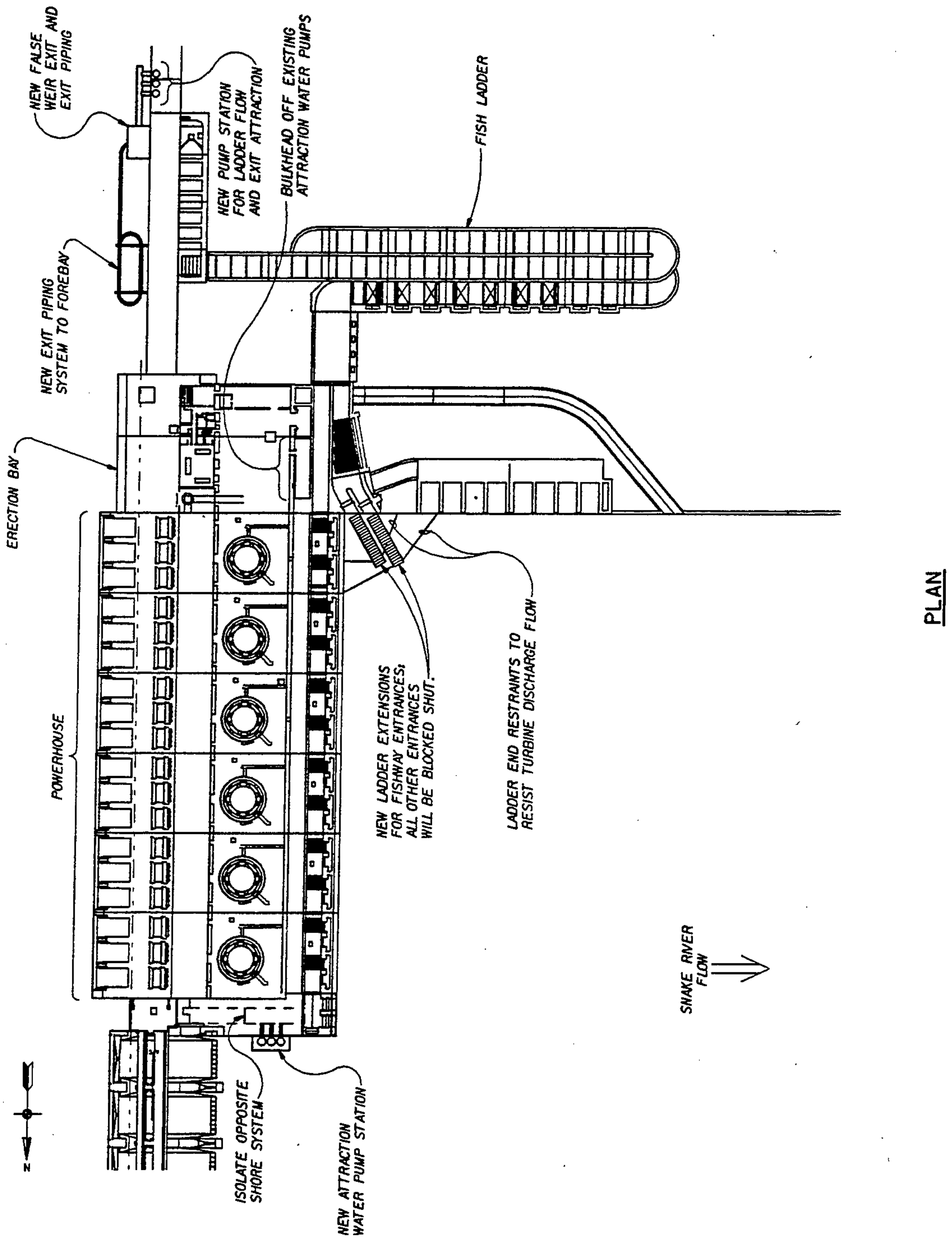

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT FISH HANDLING SYSTEM MODIFICATIONS
PLAN

Figure: C10

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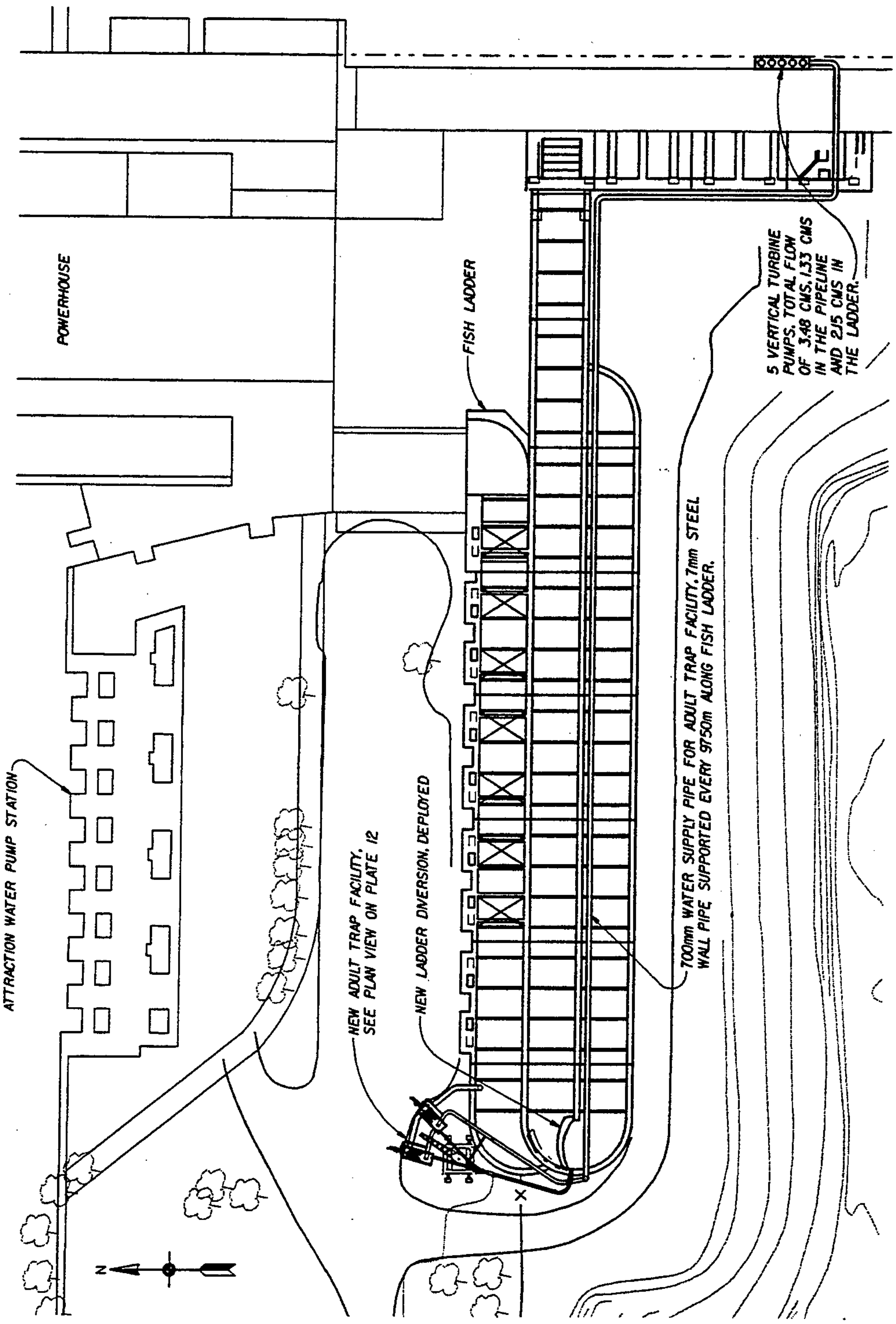

ADULT TRAP SYSTEM: OVERALL PLAN - ICE HARBOR LOCK AND DAM
(NOTE: ICE HARBOR SHOWN; LITTLE GOOSE SIMILAR)

SHEET MAIN SCALE

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT TRAP SYSTEM
OVERALL PLAN - ICE HARBOR LOCK AND DAM

Figure: C11

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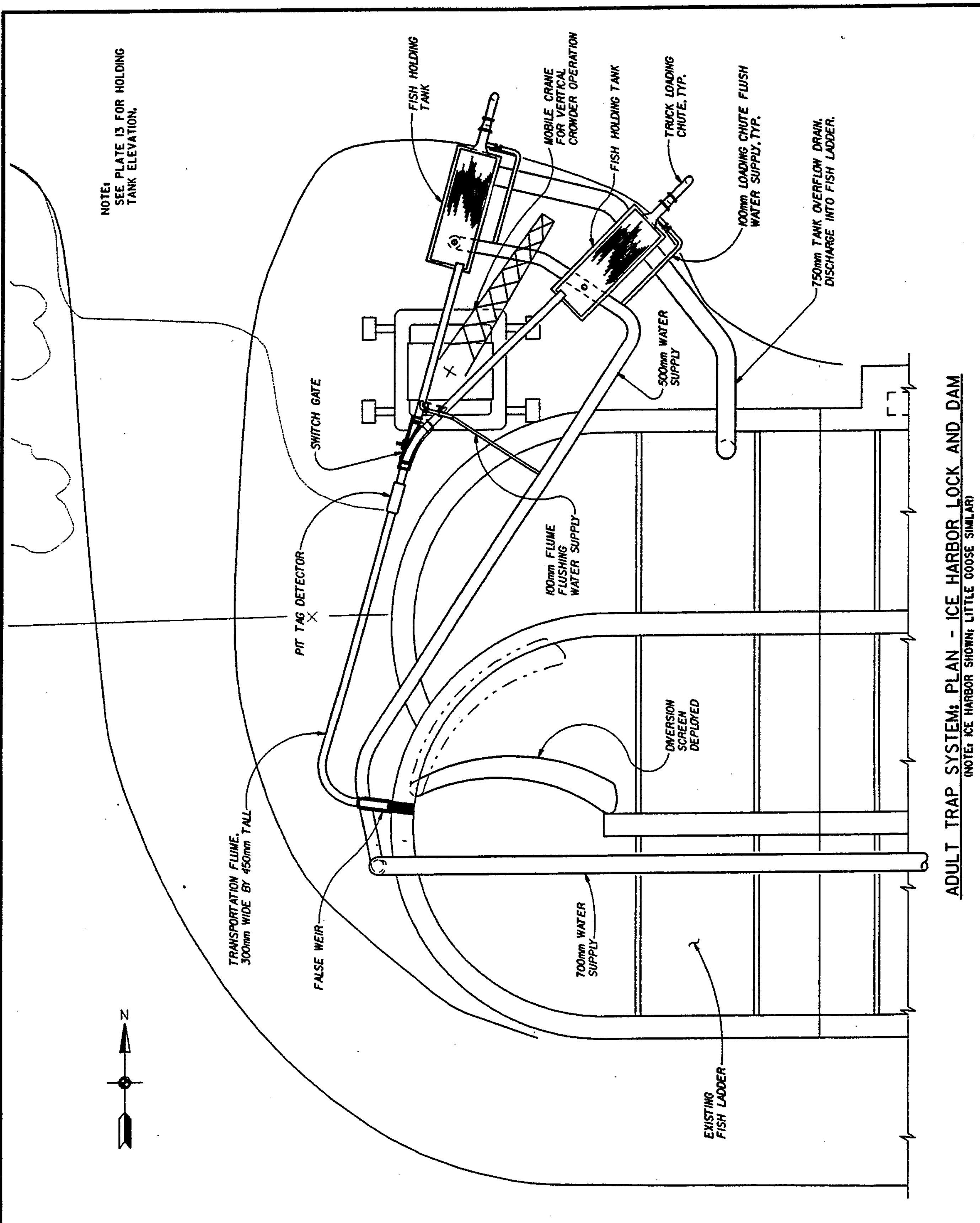

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT TRAP SYSTEM
PLAN - ICE HARBOR LOCK AND DAM

Figure:
C12

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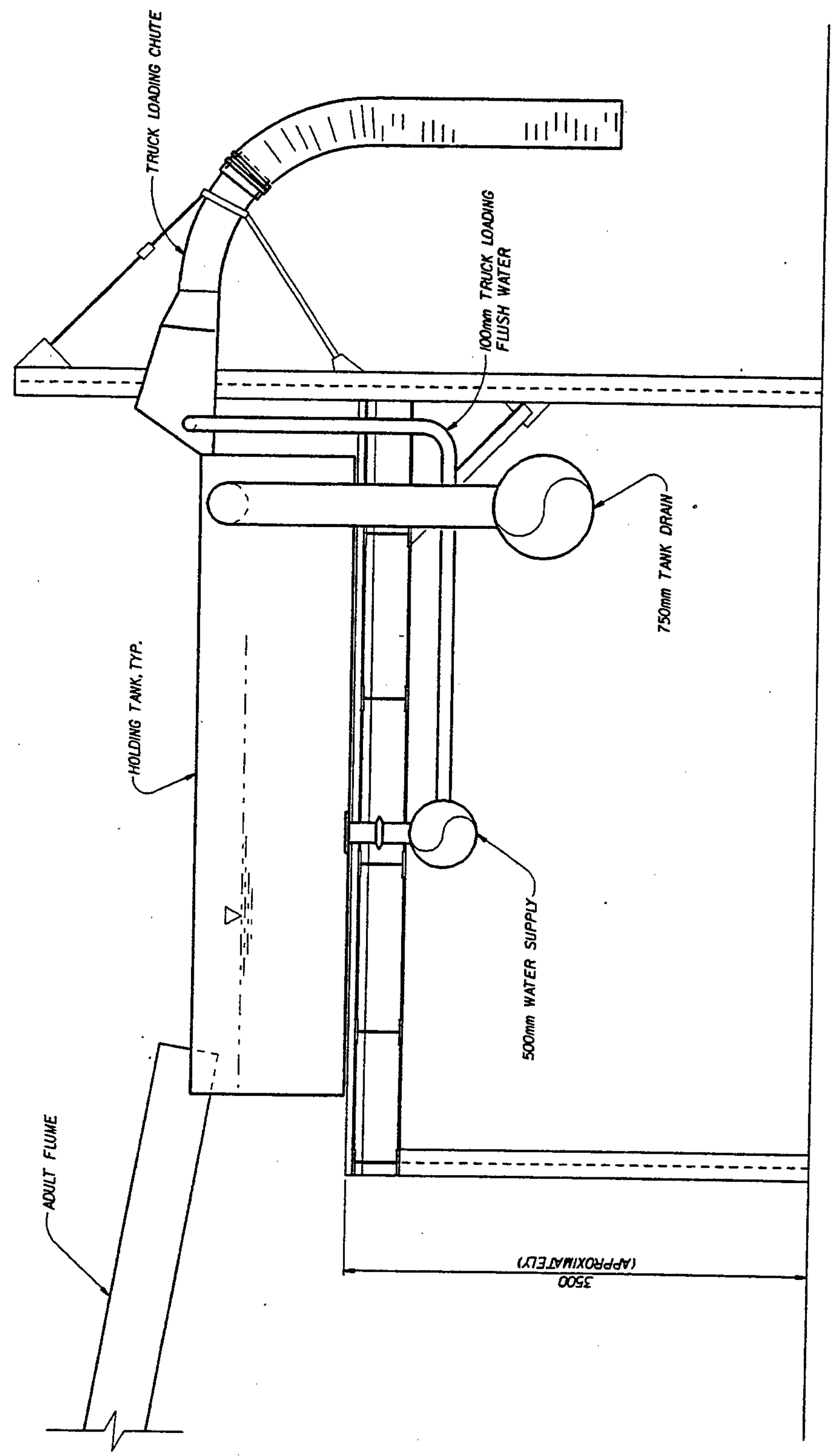
PLOT TIME: 15-OCT-1999 08:33



ADULT TRAP SYSTEM: HOLDING TANK ELEVATION - ICE HARBOR AND LITTLE GOOSE LOCK AND DAM

NOTE:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN MILLIMETERS.

US Army Corps of Engineers
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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ADULT TRAP SYSTEM
HOLDING TANK ELEVATION - ICE HARBOR AND LITTLE GOOSE LOCK & DAM

Figure: C13

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Annex D

River Channelization Plan

Annex D: River Channelization Plan

D.1 General

This river channelization plan is based on a separate report prepared for the Corps by Raytheon Infrastructure, Inc., titled *Embankment Excavation, River Channelization, and Removal of Concrete Structures* (Raytheon 1998).

This study team proposed construction of diversion levees at each project to smoothly direct the river flow into and through the new channels and around the concrete structures. The levees would be constructed of crushed rock with riprap faces placed during the period following removal and breaching of the dam embankments. The levee configurations for each of the dam sites are shown in Figures D1 through D4. A major premise in implementing a natural river state is to do so at minimal, reasonable cost. The full removal of the concrete structures would add significant cost to the project. The levees provide a means of forcing the river around the abandoned structures in a manner that allows the hydraulic performance to be analyzed and properly designed. The goal is to construct a channel that will provide acceptable fish passage for a broad range of flows and will not be damaged during high flows. Figures D1 through D4 illustrate the proposed channelization of river.

D.2 Hydraulic Considerations

Hydraulic issues are a critical factor in establishing the need for diversion levees. The primary function of the levees is to provide predictability of flow velocities and flow distribution. Without the diversion levees, flow patterns could not be predicted by analytical methods. Reverse eddies would form both upstream and downstream of the concrete structures, which would influence velocities and water surface elevations in the new channels. Predictability is necessary in the design process to guarantee performance for the range of possible conditions.

The diversion levees would be designed to be porous and to support a maximum, unbalanced head of 3 meters (m) (10 feet). The lower portion of the diversion levees would be constructed in the wet by end dumping rockfill into the river from the shore or the levee crest. The diversion levees are sized to divert a 100-year flood of 9,060 cubic meters per second (m^3/s) (320,000 cubic feet per second [cfs]) without being overtopped. The levees would also be capable of remaining in place for a flood of 11,890 m^3/s (420,000 cfs). If the 11,890-m3/s (420,000-cfs) flood overtopped the levees, there should be no appreciable damage because the levees would be essentially under balanced head. The levees would still divert most of the flow through the new channels. The diversion levees would have a crest width of 6.1 m (20 feet), and crest heights would be set 1.5 m (5 feet) above average river levels for a flow of 9,060 m^3/s (320,000 cfs). The riverside and damside slopes of both levees would be 3 horizontal (h):1 vertical (v). These slopes are very conservative. Steeper slopes may be possible after evaluation of available materials and the design of the levee section. The riverside face of both upstream and downstream levees would have 0.8 m (2.5 feet) of riprap measured normal to the slope.

Should design progress to the next stage, detailed model studies would be performed to ascertain actual flow conditions. Model studies may redefine the configuration of these levees. Studies into the performance and economics of available materials for levee construction may allow less conservative section design than proposed at this concept development stage.

D.3 Levee Design

The operational function of the diversion levees controls their design. Diversion levees would consist of rockfill overlain by a layer of riprap and would be used both upstream and downstream of the remaining concrete structures. Rockfill levees are proposed for the entire length of levee. Sheetpile cells were considered for portions of the levee that tie to the concrete structures. The sheetpile was not utilized because rockfill levees are significantly less expensive and faster to construct than the steel sheetpile levees. A typical section of the pervious diversion levee is shown on Figure D5.

The rockfill diversion levee would be constructed of porous rockfill (7.5 centimeters [cm] [approximately 3 inches]) so that water levels inside the levees (damside) would be nearly the same as the river levels outside the levees (riverside). Because the levees would be porous, they would allow some flow, but not fish, to pass through the levees and thus reduce the chance of stagnant water developing behind them. An option using a levee arrangement with a non-continuous centerline, (that is, with two parallel segments overlapping to provide a gap between them) was abandoned. While such an arrangement would be helpful in passing water to prevent stagnant conditions from developing behind the levees, it would provide a blind path or dead end that would confuse fish during migration. Since blind paths are unacceptable for fish migration, the levees alignment is continuous.

The submerged portions of the levees would be placed in the wet and, therefore, would not be compacted. The material properties of levee fill are estimated as follows. It is assumed that any fines or sand in the embankment gravel would wash out in an underwater placement.

- Unit weight of gravel/rockfill (placed underwater): 1,922 kilograms per cubic meter (kg/m^3) (120 pounds per cubic foot [pcf])
- Friction angle of gravel fill: 35 degrees
- Unit weight of riprap: 2,083 kg/m^3 (130 pcf)
- Friction angle of riprap: 40 degrees

Where rockfill levees with 3h:1v slopes join with the existing vertical concrete structure walls, a portion of the slope would protrude into the flow. This area would be heavily armored with riprap to resist the high velocities and eddies likely to be encountered at this junction.

The arrangement of the diversion cofferdams used for the original Lower Granite Dam's construction created eddies along the sections of cofferdam parallel to the lock wall. The eddies were a result of the sharp corners at the entrance to the channel and produced areas of slower velocity. The cofferdam joined the channel at right angles, and rather than forming smooth, rounded entrances, they jutted into the flow. The study team recommends that this configuration be tested in a model study to try to reproduce the turbulence and resulting slower velocities along the sides of the new channels. These slower velocities would be desirable for fish to migrate upstream, provided model studies show they do not produce flow directions and patterns that would be confusing to migrating fish. The volume of fill required for the levee at each of the dams is shown in Table D1.

Table D1. Summary of Levee Fill Material

	Barged Shotrock (m^3)	Barged Riprap (m^3)
Lower Granite	310,000	16,000
Little Goose	656,000	33,000
Lower Monumental	380,000	15,000
Ice Harbor	398,630	16,678
Total	1,743,665	66,293

D.4 Levee Fill Material

Various issues were considered to determine the most appropriate source of material for the levees. For example, at each of the dam sites, the existing embankment is on the opposite side of the river from where levee construction would begin. Therefore, temporary haul bridges, stockpiling, and double handling of embankment material would be required. In addition, embankment materials obtained from upstream of the core would be saturated. Local borrow areas on the same side of the river as levee construction (south side for all sites except Lower Monumental Dam) may be available.

The study team determined that existing embankment material should not be used for levee fill because it would involve multi-step processing. Embankment material would need to have fines to 76 millimeter (mm) (+3 inch) removed to be suitable for underwater placement. In addition there is not enough riprap of adequate size in the existing embankments, so quarried rockfill would need to be supplemented and blended with the existing material to provide suitable gradation. Therefore, use of existing embankment material for levee rockfill and riprap berms would require stockpiling, screening, double handling, and transportation across the river either by barge or bridge. This is complicated, time consuming, and expensive compared with obtaining all rockfill and riprap from one source as part of a larger rock supply operation.

Consequently, the study team assumed that all levee material – both rockfill and riprap – would come from quarries proposed for riprap production for the railroad and highway embankment protection effort that is described further in Annex F. It is most economical to take advantage of the scale of that operation to obtain the rockfill and riprap required for the channelization levees. Riprap and rockfill for channelization levees would be barged to the four respective sites and stockpiled from upstream quarries prior to the start of this project's construction.

Angular material is preferred for the underwater placement of the levee fill. Shot-rock is angular and, therefore, would be more stable for levee construction than processed embankment excavation. From a technical stability viewpoint, shotrock is preferred over alluvial embankment material because of the high angularity of the individual pieces and their ability to interlock more tightly in under water placement. Existing rock quarries near the Snake River are not as abundant as gravel pits. Haul distances for shotrock would make trucking uneconomical; therefore, barging and stockpiling shotrock is assumed for all levee fill.

Information from existing sources does not define the size of rockfill or riprap used in the existing embankment dams. However, direct observation of existing rockfill and riprap materials is the following:

- Rockfill: 0.3 to 0.6 m (1 to 2 feet) in diameter
- Riprap: 0.6 to 0.9 m (2 to 3 feet) in diameter

D.5 Channelization Levee Material Transportation

The transportation of levee materials to convenient stockpile locations is significantly impeded by removal of the embankment dam. Materials from the embankment that may be appropriate for levee construction are difficult to transport to the opposite banks because the access to the opposite shore crosses the embankment. Furthermore, access across the concrete structures cannot accommodate off-road haul vehicles or high frequency highway haul vehicle usage. Consequently, other material sources and alternate haul methods were determined to be more feasible.

Since all levee material – both rockfill and riprap – would be supplied as part of the contract for the Railroad and highway embankment protection effort, and the required loading and hauling operation would be in-place, the study team determined that transporting that material from upstream by barge would be the most cost efficient form of transportation. The study team also examined using existing bridges and local roads to transport the materials, but determined that this alternative was not feasible or cost effective.

The team determined that the closest existing bridges to each dam that would facilitate highway hauling of material were as follows:

- For Lower Granite: 38.6 km (24 miles) downstream of the project at Central Ferry
- For Little Goose: 11.3 km (6.9 miles) upstream of the project at Central Ferry
- For Lower Monumental: 26.6 km (16.3 miles) upstream of the project at Lyons Ferry
- For Ice Harbor: 13.0 km (8.1 miles) downstream of the project at State Route 12.

Using these bridges would require using the existing local roadway system that has lower load limits than for the equipment assumed. The 46-m^3 (60-cy) off-road trucks planned for embankment excavation could not be used on local roads. Truck size would be limited to approximately 15 m^3 (20 cy). Haul distances would be 32 km to 80 km (20 miles to 50 miles). The long haul distances and reduced truck size would increase unit costs for material transported over existing bridges and would be more than the cost of the material from a quarry.

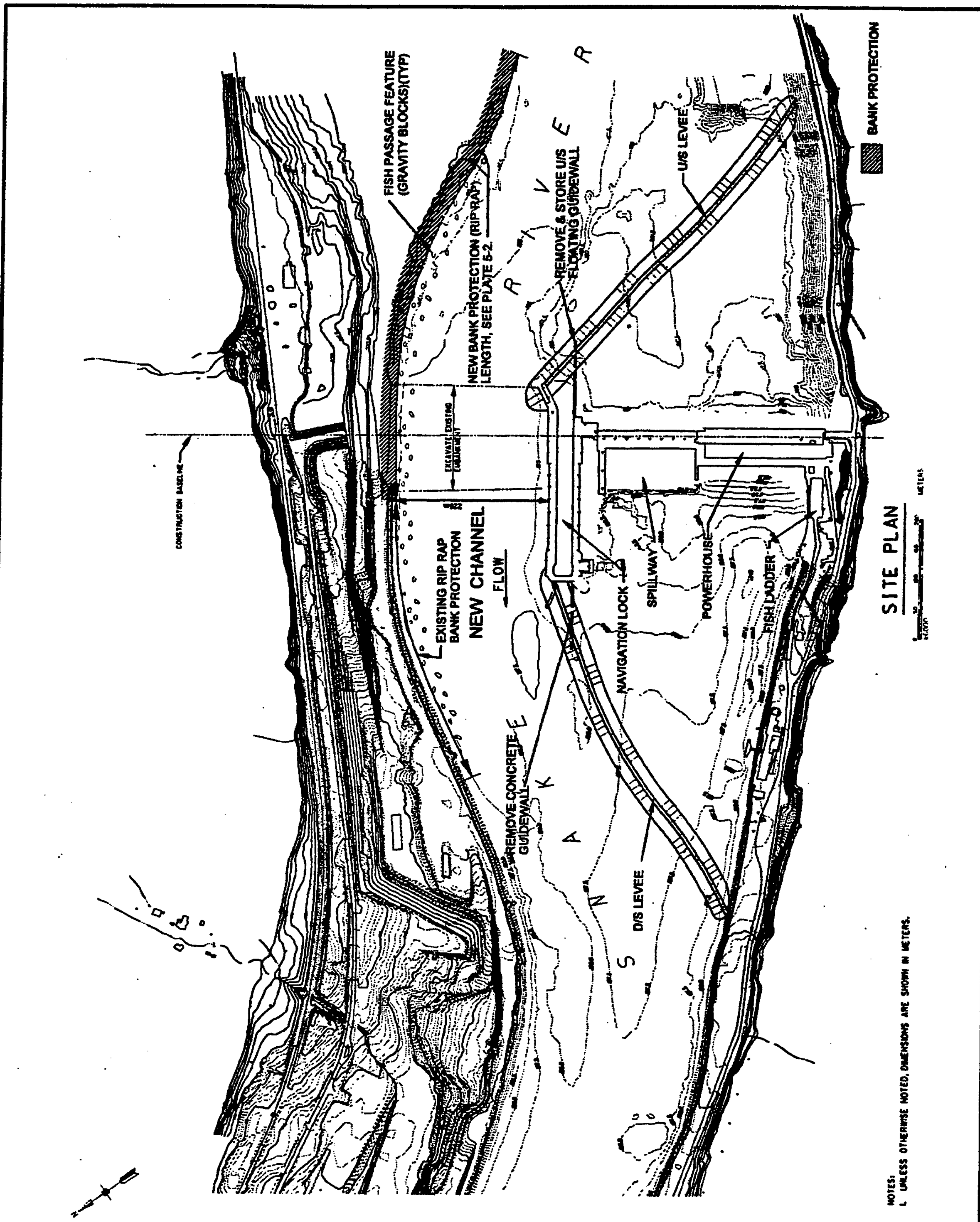
The use of temporary haul bridges or pile supported conveyor systems was determined to not be cost effective given the short duration of use, the wide range of potential river flows, and the required volume of levee materials to be transported.

D.6 Construction Sequence

Construction of the diversion levees requires controlled placement to achieve the appropriate cross section for river diversion and erosion prevention. Placement would require the use of end-dump trucks and dozers commencing construction from the shore opposite from the new channel.

Upstream diversion levee construction could begin only after the reservoir had been drawn down to 3 m (10 feet) below the proposed levee crest and the embankment had been excavated. This would be several weeks after cofferdam breaching is complete.

Construction of half of the downstream levee may begin coincident with the start of reservoir drawdown because downstream water levels stabilize at near natural levels within the first few days. However, the downstream levee cannot be closed or more than 50 percent completed until drawdown through the turbine passages is complete and the dam embankment cofferdams are breached.



US Army Corps of Engineers
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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER GRANITE

NEW CHANNEL PROJECT ARRANGEMENT - PLAN

Figure:
D1

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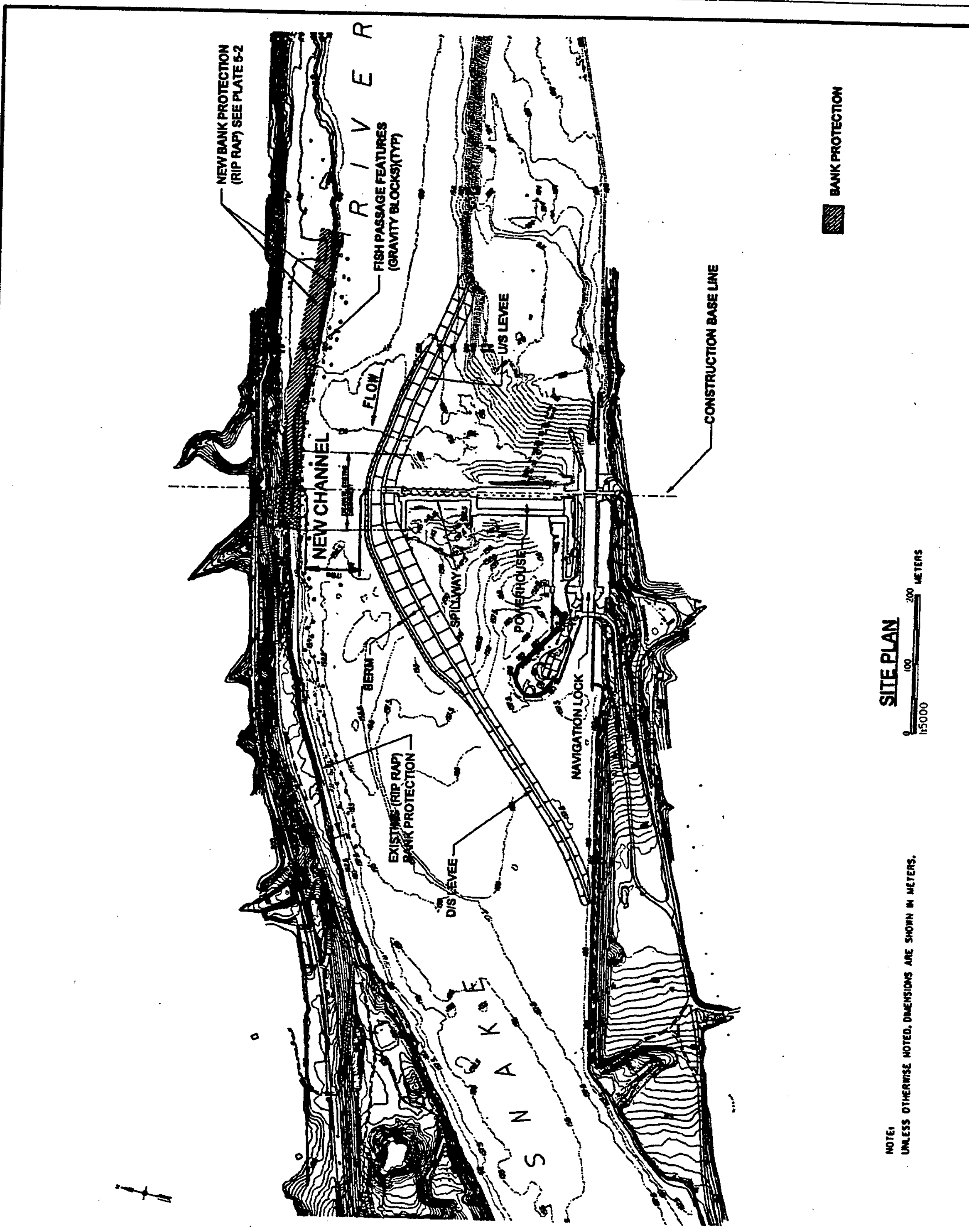

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
LITTLE GOOSE
NEW CHANNEL PROJECT ARRANGEMENT - PLAN

Figure:
D2

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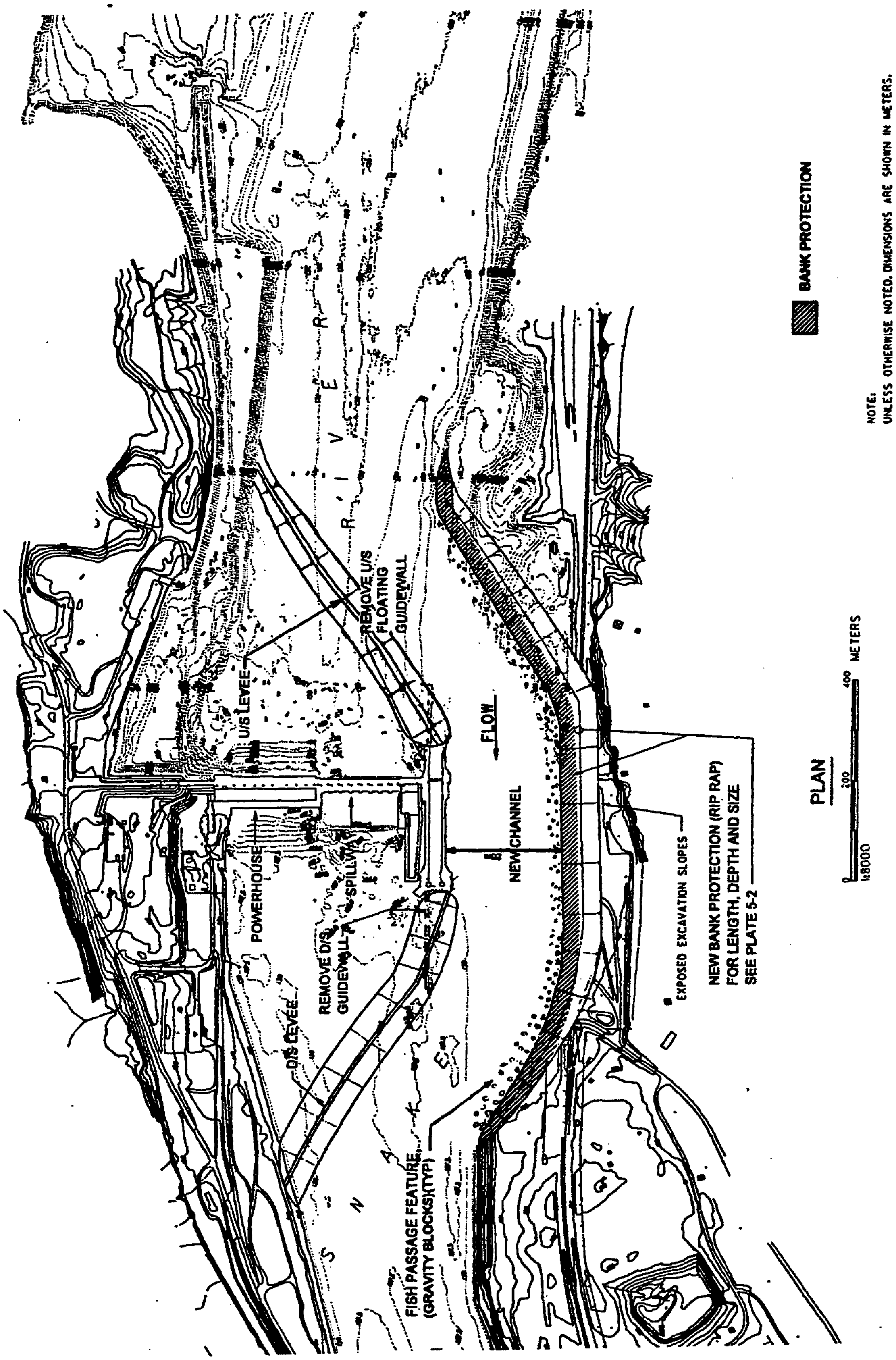
SHEET MAIN SCALE



US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
LOWER MONUMENTAL
NEW CHANNEL PROJECT ARRANGEMENT - PLAN

Figure:
D3

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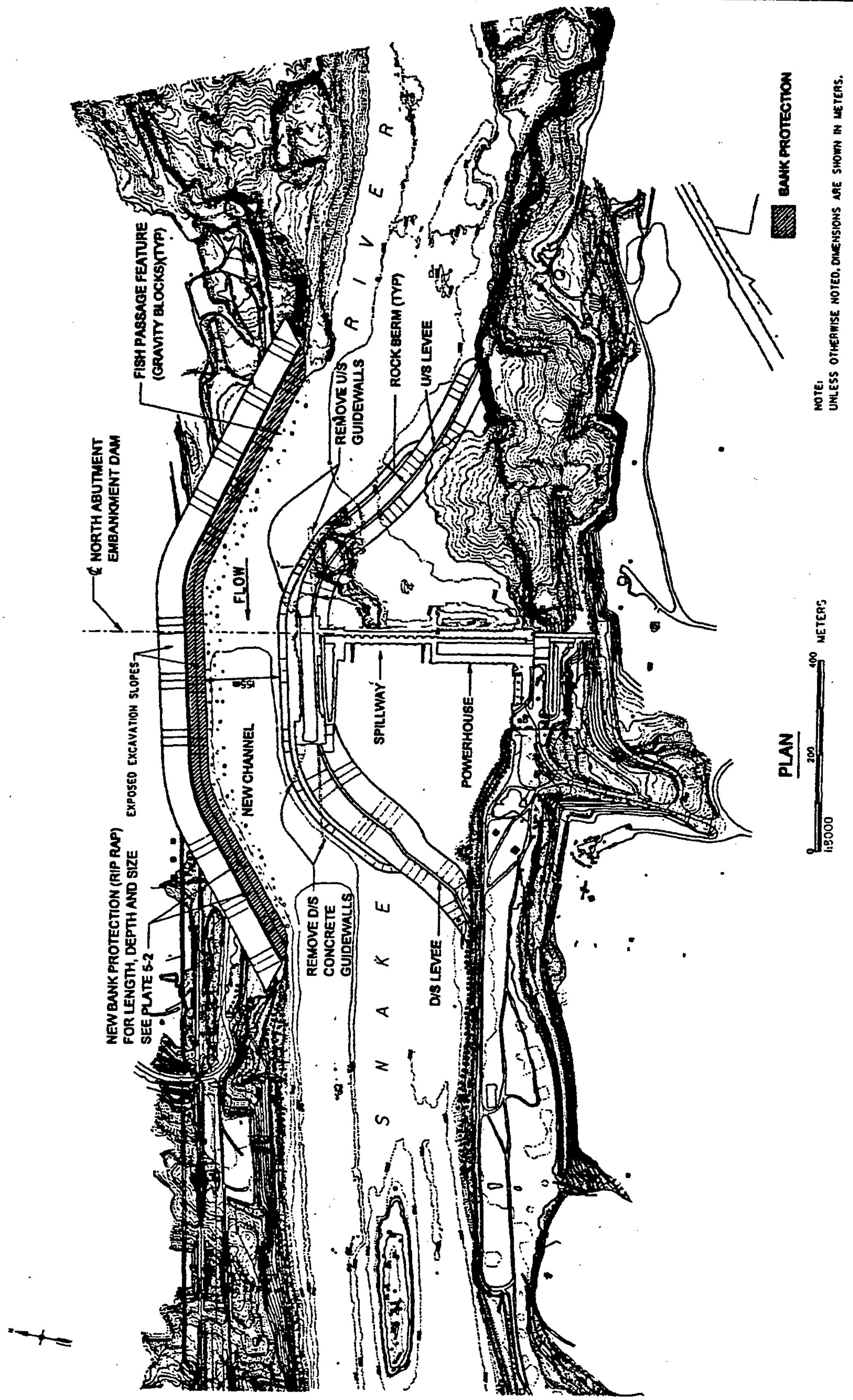

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

ICE HARBOR

NEW CHANNEL PROJECT ARRANGEMENT - PLAN

Figure: D4

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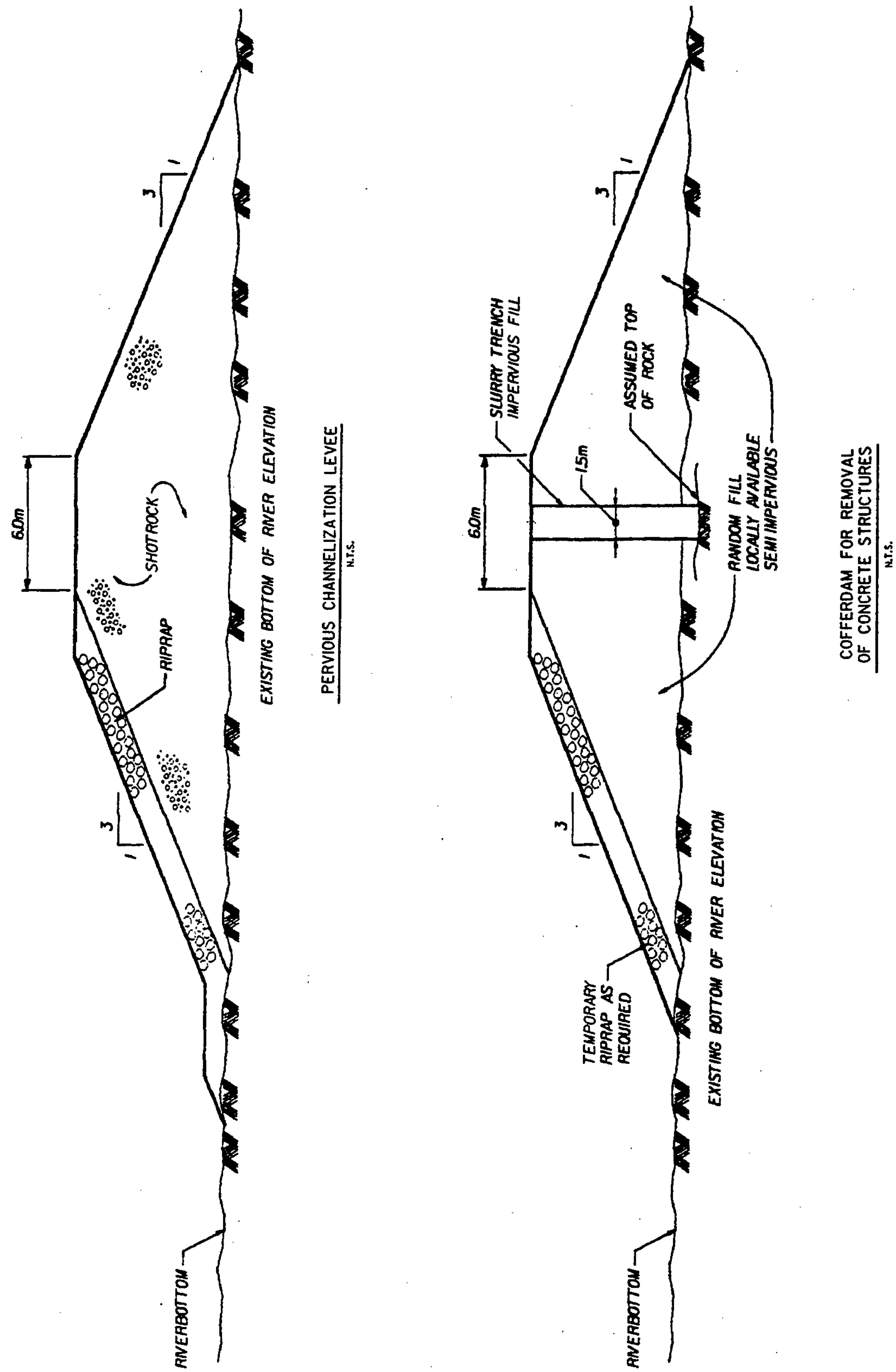

NOTES:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN METERS.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
EMBANKMENT EXCAVATION AND RIVER CHANNELIZATION
TYPICAL LEVEE SECTIONS

Figure: D5

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Annex E

Bridge Pier Protection Plan

Annex E: Bridge Pier Protection Plan

E.1 General

A field survey, performed in August 1995, identified 25 bridges that could be affected by permanent drawdown of the four lower Snake River reservoirs. These highway and railroad bridges were evaluated to determine the adequacy of the existing bridge foundations and abutment protection to resist post-drawdown flood scouring to natural stream levels. Of the 25, all but 2 required some degree of protection. These structures included bridges at several locations crossing the Snake and Clearwater rivers and on tributaries to the Snake, adjacent to the reservoirs. The selected bridge identifications and locations are shown on Table E1.

Typical bridge supports for these structures include concrete seals and concrete footings, steel or wood piles, pile bents, and other cofferdam footing foundations for piers. After reservoir drawdown, existing bridge pier foundations may be destabilized or undermined by the following conditions:

- Long-term stream degradation and aggradation
- Local scour due to an accumulation of debris which restricts flow and increases scour severity
- Stream instability, possibly due to the migration of the river channel
- Erosion of approach embankments adjacent to abutments.

E.2 Standard Modifications

The existing road and railroad bridges along the lower Snake River are to remain functional under the proposed reservoir drawdown to natural streambed elevations. Some modifications to the existing bridge piers and abutments may be required where flow depths and velocities change and thus affect local river bottom deposits and native soils. Modifications may be required for older bridges that were in place prior to reservoir impoundment, as these bridges may not be adequately protected against scour when evaluated by modern scour analysis techniques. The two categories of bridge pier modification are Abutment Erosion Protection and Bridge Pier Foundation Reinforcement.

E.2.1 Abutment Erosion Protection

The study team's key concern was protecting existing bridge support structures from flowing water erosion and from undermining after drawdown by potential flood events. In general, the study team determined that abutment erosion protection should be placed between the elevations of the 500-year flood event (flows vary depending upon location), and normal low flow (820 m^3/s) in areas where available information indicates that adequate protection (rock or riprap) does not currently exist on exposed banks and abutments.

The size of riprap to be used for erosion protection can be determined from established tables (Corps of Engineers, Waterways Experiment Station, Hydraulic Design Chart 712-1), which relate river flow velocity, stone weight, and the diameter of the riprap required. The river flow velocity is determined from river profile and cross-section elevations in conjunction with using Hydrologic Engineering Center's computer program entitled "Water Surface Profiles," version 4.6.2, (commonly referred to as "HEC-2") or its computer program entitled "River Analysis System," version 2.1 (commonly referred to as "HEC-RAS™"). The study team assumed a minimum stone weight of 2,162 kilograms per cubic meter (kg/m^3) (135 pounds per cubic foot [pcf]) for the riprap sources available on the lower Snake River. These values were used to determine the diameter, D_{50}, of the riprap to be placed on the slope to be protected. The subscript in D_{50} refers to the percent of rock in which the diameter is less than the size

Table E1. Bridge Types and Locations

Bridge Name	Type	Location
Joso River	Railroad	Snake River/Lower Monumental Reservoir at River Kilometer 94.1
Lyons Ferry	Highway	Snake River/Lower Monumental Reservoir at River Kilometer 95.3
Snake River	Railroad	Snake River/Lower Monumental Reservoir at River Kilometer 99.4
Central Ferry	Highway	Snake River/Little Goose Reservoir at River Kilometer 134.1
Red Wolf	Highway	Snake River/Lower Granite Reservoir/Northwest Clarkston at River Kilometer 221.1
Snake River (Old US 12)	Highway	Snake River/Lower Granite Reservoir/Clarkston at River Kilometer 224.5
Southway	Highway	Snake River/Lower Granite Reservoir/Clarkston at River Kilometer 227.6
Lewiston (Camas Prairie)	Railroad	Clearwater River/Lower Granite Reservoir at River Kilometer 0.6
Clearwater Memorial	Highway	Clearwater River/Lower Granite Reservoir at River Kilometer 3.4
Tributary Bridge No. 1 (Steptoe Canyon)	Hwy. & RR	Lower Granite Reservoir at River Kilometer 206.0
Tributary Bridge No. 2 (Nisqually John Canyon)	Hwy. & RR	Lower Granite Reservoir at River Kilometer 198.2
Tributary Bridge No. 3 (Yakawawa Canyon)	Hwy. & RR	Lower Granite Reservoir at River Kilometer 189.6
Tributary Bridge No. 4 (Keith Canyon)	Hwy. & RR	Lower Granite Reservoir at River Kilometer 188.8
Tributary Bridge No. 5 (Wawawai Canyon)	Railroad	Lower Granite Reservoir at River Kilometer 178.1
Tributary Bridge No. 6 (Buck Canyon)	Hwy. & RR	Lower Granite Reservoir at River Kilometer 175.8
Tributary Bridge No. 7	Hwy. & RR	Lower Granite Reservoir at River Kilometer 198.6
Tributary Bridge No. 8	Railroad	Little Goose Reservoir at River Kilometer 124.7
Tributary Bridge No. 9	Railroad	Little Goose Reservoir at River Kilometer 121.2
Tributary Bridge No. 10	Railroad	Little Goose Reservoir at River Kilometer 118.9
Tributary Bridge No. 11	Railroad	Little Goose Reservoir at River Kilometer 116.2
Tributary Bridge No. 12	Highway	Little Goose Reservoir at River Kilometer 133.7
Tributary Bridge No. 13	Highway	Little Goose Reservoir at River Kilometer 167.0
Tributary Bridge No. 14	Railroad	Little Goose Reservoir at River Kilometer 149.9
Tributary Bridge No. 15	Railroad	Little Goose Reservoir at River Kilometer 147.6

noted. The thickness of the bank or abutment protection is chosen as two times the D_{50} of the riprap (i.e., if the D_{50} is determined to be 0.3 meters [m], the thickness of the bank protection would be 0.6 m).

The lateral extent (parallel to stream flow) of the bank protection is determined from U.S. Department of Transportation criteria set forth for protection of bridge piers (WDOT 1990). This criteria requires a riprap mat width of at least two times the pier width measured from the face of the pier in the upstream and downstream directions. This results in a mat equal to four times the width of the pier plus the length of the pier. If the area to be riprapped does not include a bridge pier, then the nearest bridge pier is used

as a reference to size the area that needs the riprap protection. It should also be noted that the riprap mat is symmetrical with the centerline of the bridge and abutment in question.

Placing additional riprap armor surrounding the bridge support interface with the native soils or rock would provide a cost-effective modification to reduce the effects of scour. Figure E1 illustrates a typical abutment protection modification.

E.2.2 Bridge Pier Foundation Reinforcement

Drawdown also could require protection or reinforcement of existing bridge pier foundations. Reinforcement of the bridge support foundations would be necessary because all streambed material in the new scour prism could be removed and, thus, not be available for bearing or lateral support of the piers.

The study team made the following assumptions in evaluating remedial measures for the piers:

- Where the calculated scour depth is above the top of the existing footing, no protection would be required.
- Where footings of piers are founded on rock, no additional protection of the footing would be required. The team assumed that existing concrete footings are founded on competent rock, resistant to scour erosion.

Where the calculated scour depth for a concrete footing resting on soil falls between the top of the footing and the bottom of the footing, interlocking steel sheetpiles would be driven in a circular cell configuration to 2 m below the calculated scour depth. This additional margin of safety would be provided because of the uncertainties in the scour prediction methods.

Where the calculated scour depth for a concrete footing resting on soil falls below the bottom of the footing, interlocking steel sheetpiles would be driven to at least 2 m below the calculated scour depth. If the calculated scour depth is at or near the level of bedrock, the circular cell sheetpiles would be driven to refusal into bedrock.

For all interlocking steel sheetpile installations, the annular space between the new sheetpiles and the existing concrete structure would be filled with concrete to serve as a cap protection from erosion. This cap would be at least 0.5 m thick. In some cases, excavation of up to 1 m depth of river bottom material, between the pier foundation and the surrounding sheetpile, would be needed to place the concrete cap. Figure E2 provides a typical treatment of bridge pier foundations.

Generally, for all interlocking circular cell steel sheetpile installations, the top of the steel sheetpile wall would be established by the higher of two elevations: 1) the elevation of the streambed, or 2) the construction season low water surface level. The top of the steel sheetpile circular cell wall would be cut off 0.5 m above the higher of the two. This removal of excess sheet pile material, which may have to be accomplished underwater, would not adversely affect scour protection or hinder river navigation.

E.3 Evaluation of Modifications

Conceptual modifications for each site-specific existing bridge structure were selected, based on flow parameters and scour analyses as they applied to that bridge's pier configurations. Potential scour, evaluated at flows for the 500-year flood event for each bridge, was estimated to range from 2.75 m to 8.2 m (9 feet to 27 feet) below streambed elevation. Calculations for each bridge analyzed are given in Appendices C and G of a separate report titled *Lower Snake River Reservoir Stabilization Plan* (Raytheon 1997).

The following two subsections describe the methodology and river flow values that were used to determine the modifications needed for each bridge analyzed.

E.3.1 Scour Evaluation Methodology

Calculations of scour depth for bridge piers were performed for each of the bridge sites. These calculations were performed using the HEC-RAS™ program for the 500-year flood event to determine flow characteristics at each site during this event. The HEC-RAS™ software program, along with river profile and cross-section elevations, provides information on total flow, velocity, water surface elevations, and channel dimensions for all data stations on the lower Snake River during this event. These hydraulic calculations were performed for river water surface elevations resulting from the removal of the dams and the return to the original streambed conditions prior to reservoir impoundment. Because of the long span between bridge piers and the large size of the existing Snake River valley, contraction scour was considered to be minimal and, therefore, was ignored. Consequently, total scour was estimated by calculation of local scour only. Contraction scour also was not considered by the Washington State Department of Transportation (WSDOT) in the calculation of total scour for two bridges (Snake River and Central Ferry Highway bridges) on the lower Snake River inspected by WSDOT. Scour depths were estimated from channel bottom elevations shown on as-built drawings of the existing bridges. Because of the high reservoir pools and the controlled velocities resulting from the construction and operation of the dams, it is likely that the river has aggraded and accumulated sediment, resulting in higher stream bottom elevations than originally encountered at the bridge piers.

The methodology used in determining potential scour for all bridge structures was based on that contained in *Evaluating Scour at Bridges, Hydraulic Engineering Circular No. 18* (WDOT 1990). This methodology is used throughout the country when performing bridge scour evaluations.

E.3.2 River Flows Evaluated

Bridge scour studies performed throughout the U.S. have been largely based on 500-year flood flows. To be consistent with these state and Federal criteria, calculations of scour depths for this study are based on the 500-year flood event. The design flow information is summarized as follows:

Table E2. Flow Conditions For River Sections

River Section	500-yr Flow	100-yr Flow	Normal Low Flow
Snake River - above confluence with Clearwater River:	8,350 m^3/s (295,000 cfs)	7,415 m^3/s (262,000 cfs)	820 m^3/s (29,000 cfs)
Snake River - below confluence with Clearwater River	10,190 m^3/s (360,000 cfs)	9,050 m^3/s (320,000 cfs)	820 m^3/s (29,000 cfs)
Clearwater River:	1,840 m^3/s (65,000 cfs)	1,640 m^3/s (58,000 cfs)	820 m^3/s (29,000 cfs)

E.3.3 Results for Each Bridge

The study team identified various configurations of foundation modifications for the selected bridges within the project area. These are summarized below along with the results of the scour evaluation.

Joso River Railroad Bridge

The Joso River Railroad Bridge is located on the Snake River at river kilometer 94.1. The bridge is supported on multiple concrete piers founded on concrete footings that extend to bedrock. The potential scour was evaluated for a 500-year flood event flow of 10,190 m^3/s (360,000 cfs) with a corresponding water surface elevation of 152.5 m (500.4 feet). Estimated scour depths were 8.2 m (27 feet). Since the

piers are founded on bedrock and, therefore, are assumed to be resistant to erosion, no additional bridge pier modifications would be required. Abutment protection on the south abutment would be required for the fluctuating water surface.

Lyons Ferry Highway Bridge

The Lyons Ferry Highway Bridge is located on the Snake River at river kilometer 95.3. The bridge is supported on multiple concrete piers founded on concrete footings that terminate on soil or bedrock. The potential scour for this bridge was evaluated for a 500-year flood event flow of 10,190 m^3/s (360,000 cfs) with a corresponding water surface elevation of 153.0 m (502.1 feet). Estimated scour depths were 5.5 m (18 feet). There are five piers outside the river flow path that, therefore, do not require treatment. Since the three existing piers are founded on bedrock and, therefore, are assumed to be resistant to erosion, no additional modifications would be required. The remaining piers (number 4 and 8), as well as both abutments, would require protection.

Snake River Railroad Bridge

The Snake River Railroad Bridge is located on the Snake River at river kilometer 99.4. The bridge is supported on multiple concrete piers founded on concrete footings on bedrock. The potential scour was evaluated for a 500-year flood event flow of 10,190 m^3/s (360,000 cfs) with a corresponding water surface elevation of 156.6 m (513.9 feet). Calculated scour depths were 8.0 m (26.3 feet). Because all of the piers are founded on bedrock and, therefore, are assumed to be resistant to erosion, no additional modifications would be performed at this bridge location.

Central Ferry Highway Bridge

The Central Ferry Highway Bridge is located on the Snake River at river kilometer 134.1. The bridge is supported on multiple concrete piers founded on concrete footings and pile-supported concrete footings. The potential scour was evaluated for a 500-year flood event flow of 10,190 m^3/s (360,000 cfs) with a corresponding water surface elevation of 176.6 m (579.5 feet). Calculated scour depths were 5.4 m (18 feet) for the shallow piers (piers 2 and 7) and 6.4 m for the larger, deeper piers (piers 3 through 6). This condition could undermine the foundations of the piers and, thus, would require modifications at each of the pier locations. See Figures E3 and E4 are pier modifications for Central Ferry Highway Bridge and typical for most other bridges. Abutment protection against erosion would also be needed.

Red Wolf Bridge

This bridge is located on the Snake River at river kilometer 221.1 and crosses the Snake River at the northwest part of Clarkston. Concrete piers founded on concrete footings support the bridge. Piers 3 and 4 have rock anchors in their foundations that extend to bedrock. The remaining piers (piers 2 and 5) are founded below the level of the existing streambed.

Scour calculations were performed for 500-year flood event flows of 10,190 m^3/s (360,000 cfs) corresponding to a water surface elevation of 221.6 m (727 feet). With these criteria, the projected scour depth is 6.7 m (22 feet). This condition could undermine the foundations for piers 2 and 5. To provide adequate protection, sheetpile isolation of piers 2 and 5 and riprap for abutment protection would be required.

Snake River Highway Bridge (Route 12)

The Snake River Highway Bridge is located on the Snake River just upstream from its confluence with the Clearwater River. The existing bridge is supported by multiple concrete piers founded on concrete footings that are either on bedrock or on piles driven into bedrock. Scour potential was calculated based

on a 500-year flood event flow of 8,350 m^3/s (295,000 cfs) and a water surface elevation of 223 m (733 feet). Based on recent soundings (WDOT 1990), it appears that aggradation of streambed materials has occurred in the vicinity of the bridge since original construction. For scour analysis purposes, the study team assumed that drawdown conditions would result in original streambed elevations. Therefore, potential scour was estimated to be 5.2 m (17 feet) for the largest piers supporting the lift section of the bridge and about 2.7 m to 3.0 m (9 feet to 10 feet) for the remaining smaller piers. This condition could undermine the foundation for the larger pier footing No. 3, supported on piles, but not those of the smaller piers in the more shallow water. Modifications, including driving interlocking steel sheetpiles to below the depth of calculated scour, would be required for Pier No. 3, even though it was in place prior to reservoir impoundment. Riprap abutment protection would also be required.

Southway Highway Bridge

The Southway Highway bridge is located approximately 3.2 kilometers (2 miles) south of the confluence of the Snake and Clearwater rivers. The Southway bridge is supported by multiple concrete piers with the base of the concrete footings extending approximately 5.2 m (17 feet) below the riverbed. These footings appear to extend to bedrock. Based on a 500-year flood event flow of 8,350 m^3/s (295,000 cfs) and a water surface elevation of 225 m (740 feet), the potential scour depth is approximately 4 m (13 feet). Since the bedrock is assumed to be resistant to erosion, the footings would not require additional modifications. Riprap abutment protection would be required.

Lewiston (Camas Prairie) Railroad Bridge

The Lewiston (Camas Prairie) Railroad Bridge is located on the Clearwater River in proximity to the confluence to the Snake River. The bridge is supported by multiple concrete piers with all except two of the piers founded on rock. Pier No. 3 is supported by a grouted concrete column surrounded by a steel sheetpile cofferdam extending to rock. Pier No. 4 is supported by piles extending through riverbed materials and founded on rock. Project drawings show the pilings ranging in length from about 6.1 m to 7.6 m (20 feet to 25 feet). Based on a 500-year flood event flow of 1,840 m^3/s (65,000 cfs) in the Clearwater River, the scour potential is calculated to be approximately 3.4 m (11 feet). This would expose the pilings on the new pier No. 4 and subject the pilings to potential lateral motion and failure. Therefore, modifications would be required to protect Pier No. 4 from scour erosion. No abutment protection would be required.

Clearwater Memorial Highway Bridge

The Clearwater Memorial Highway Bridge is located on the Clearwater River approximately 2.4 kilometers upstream from the confluence of the Snake and Clearwater rivers. The bridge is supported on multiple piers founded on footings. The potential scour was evaluated at a 500-year flood event flow of 1,840 cubic meters per second (m^3/s) (65,000 cubic feet per second [cfs]) and a water elevation of 223.5 m (733 feet) based on Corps river profiles. Estimated potential scour depths were 2.75 m (9 feet). This depth could undermine piers not founded on rock, even though this bridge was in place prior to reservoir impoundment. Therefore, pier modifications would be required on Piers 2 through 7. Piers 8, 9 and 10 are founded on rock and need no protection. The existing ground surface of Piers 1 and 11 is at or above the 500 year flood level and, therefore, do not require protection. No abutment protection is required.

Tributary Bridges

There are 15 bridges located across various tributaries draining into the Snake River on the Lower Monumental, Little Goose, and Lower Granite reservoirs. The bridge axes are generally oriented roughly parallel to the flow of the Snake River. Present inundation of the area by the reservoirs create a slack water condition that would be eliminated under drawdown to natural streambed elevations. The 500-year

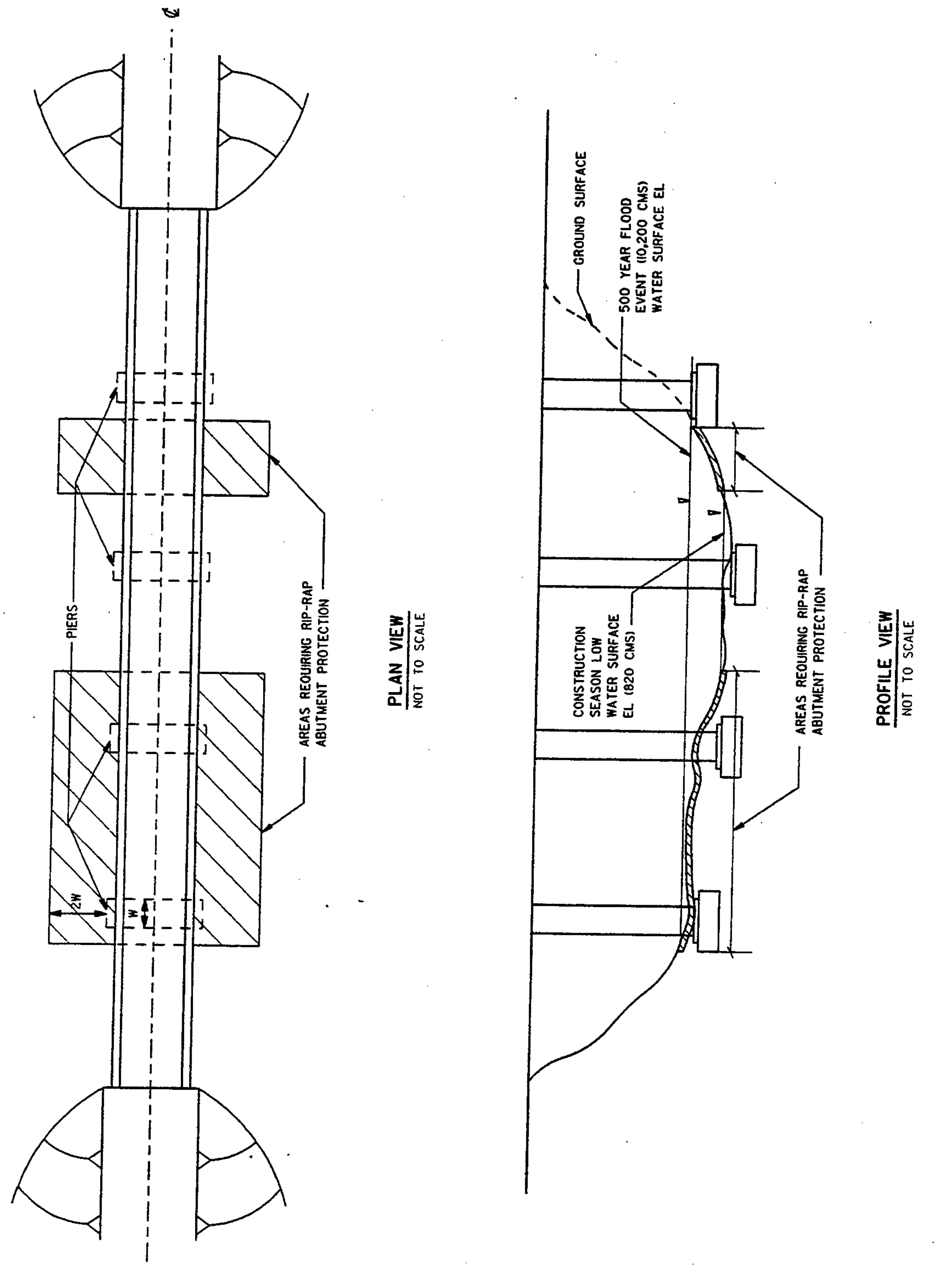
flood event at 10,190 m^3/s (360,000 cfs) has corresponding Snake River water surface elevations putting the bridge channel elevations higher than the 500-year flood event. Any potential scour to the bridge piers on tributaries to the lower Snake River reservoirs would originate from tributary flows, not from Snake River flows. In general, highway bridges are supported only at the abutments with no intermediate piers or piles, and abutments are heavily armored by riprap. Typically, one-to-three intermediate steel "H" piles, as well as the two end abutments support the railroad bridges.

Because stream flow data and elevations corresponding to the 500-year flood events on tributaries to the lower Snake River are not available, a typical riprap protection measure was established to represent the protection measure needed to safeguard the 15 tributary bridges. This typical treatment measure is shown in Figure E1. An average span bridge crossing of the 15 tributaries along the lower Snake River reservoirs was selected to represent the additional riprap needed to armor abutment and central piers from tributary flows.

E.4 Construction Scenario

The nature of the bridge pier and abutment modifications required to maintain the safe functioning and integrity of existing bridges on the lower Snake River reservoirs is such that each bridge could be considered a separate project and should be performed after drawdown of the reservoirs. Only two (Snake River Highway, Route 12; and Lewiston Railroad, Camas Prairie) of the eight bridges require a barge-supported pier protection installation. The other bridge modifications can all be performed from land-based construction access, assuming mid-December to mid-March time periods. The work could be combined in one contract, or separated into three (one for each reservoir).

For each bridge requiring modifications, a construction schedule was prepared based on a standardized list of construction activities. This base list includes all major functions necessary to complete the entire menu of construction work that was identified for all bridge work considered. Quantities for each site-specific bridge and its modification requirements were added to the schedule. Durations for completion of the construction activities were then calculated based on selected productivity rates and the number of construction crews to perform the work tasks. Construction durations range from 16 workdays for the Joso River Bridge modifications to 47 workdays for the Central Ferry Highway Bridge modifications, which include 22 workdays for driving steel sheetpiles. Annex W summarizes the activities and related timeframe required to implement design and construction of these bridge pier protection measures. The timeframe for this work spans a period of 2 or 3 years.



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

BRIDGE ABUTMENT PROTECTION

Figure: E1

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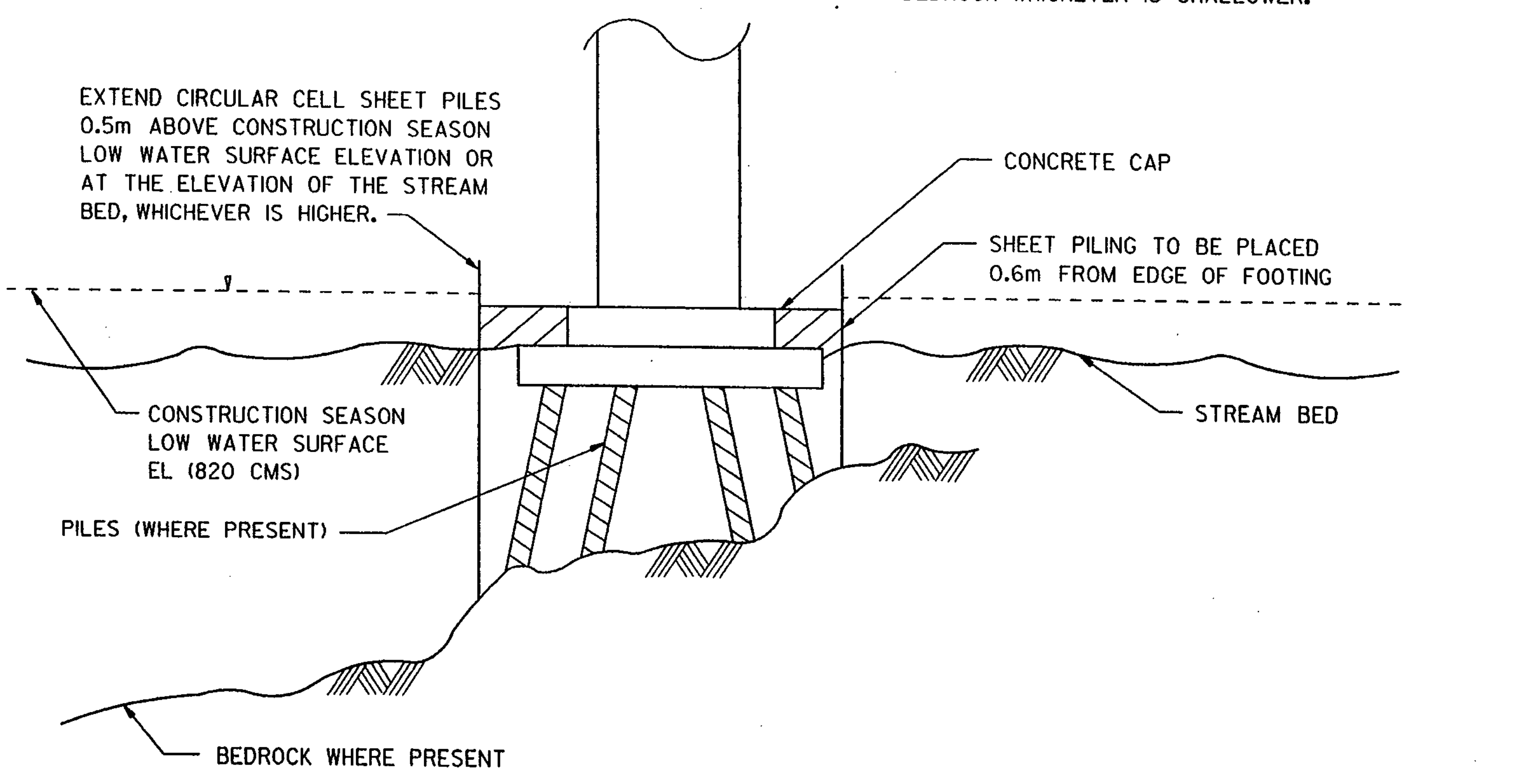

PROFILE VIEW

NOT TO SCALE

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

BRIDGE PIER PROTECTION

Figure: E2

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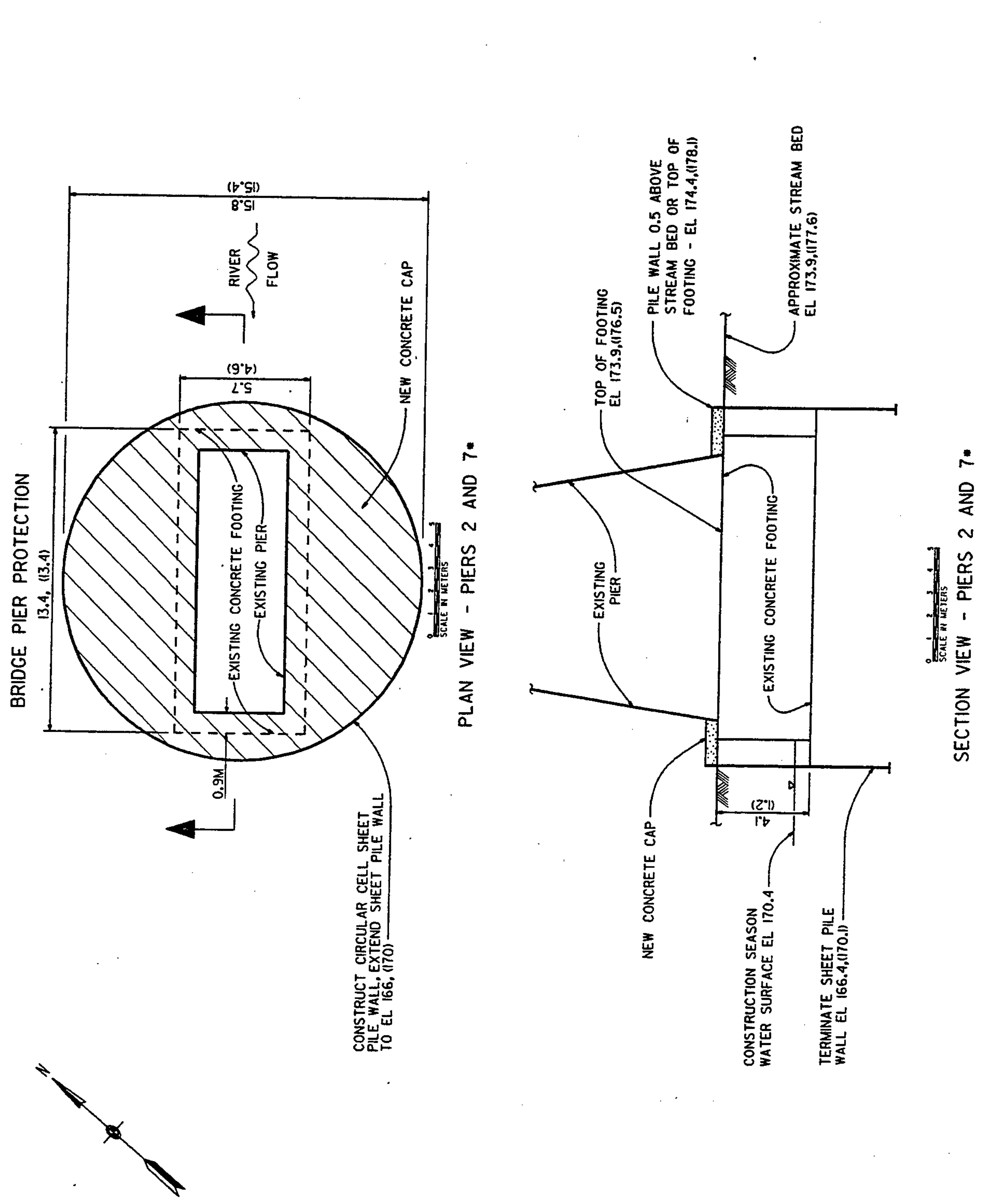

NOTE:
UNLESS OTHERWISE SHOWN, ALL DIMENSIONS ON THIS SHEET ARE IN METERS.

* DIMENSIONS FOR PIER 7 ARE SHOWN IN PARENTHESES. PIER 7 IS PILE SUPPORTED.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

CENTRAL FERRY HIGHWAY BRIDGE PIERS 2 AND 7
PLAN AND SECTION

Figure: E3

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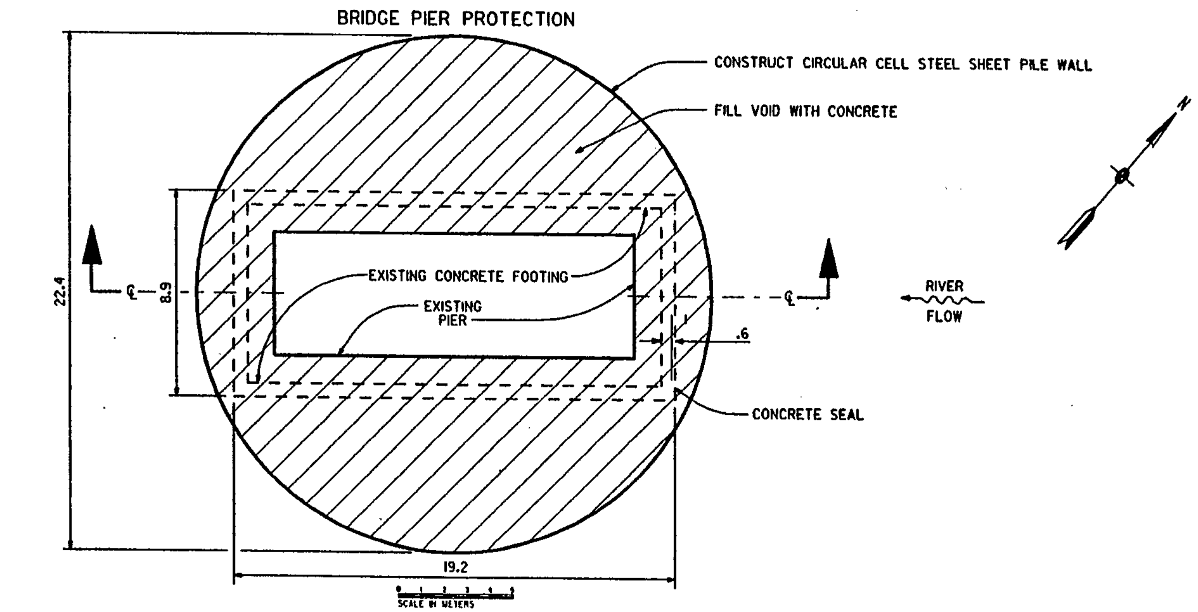
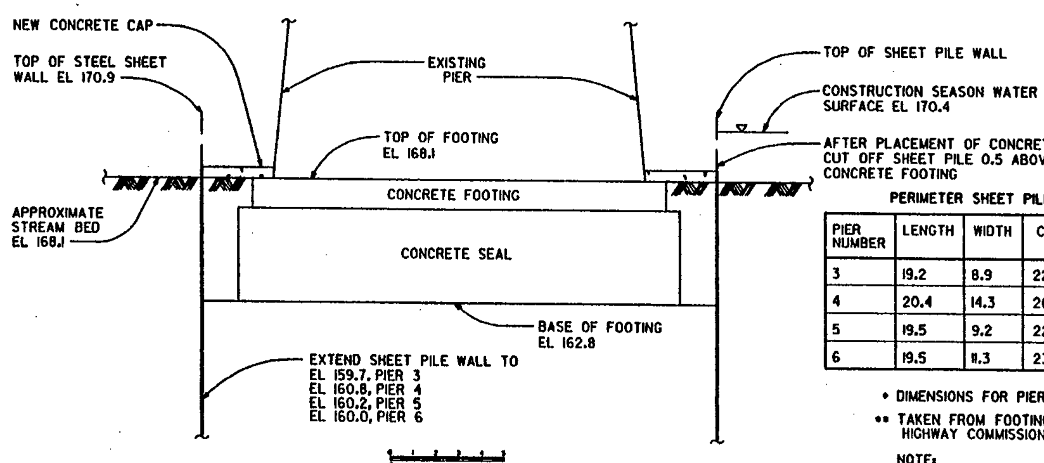
BRIDGE PIER PROTECTION



PLAN VIEW (PIER 3•)



SECTION VIEW - PIER 3•

PERIMETER SHEET PILE DIMENSIONS FOR CENTRAL FERRY HIGHWAY BRIDGE ••

PIER NUMBER	LENGTH	WIDTH	CIRCULAR CELL DIAMETER	TOP OF FOOTING EL	TOP OF SHEET PILE WALL - EL
3	19.2	8.9	22.4	168.1	170.9
4	20.4	14.3	26.2	169.2	170.9
5	19.5	9.2	22.8	168.6	170.9
6	19.5	11.3	23.8	168.4	170.9

• DIMENSIONS FOR PIERS 4,5, AND 6 ARE SHOWN IN THE ACCOMPANYING TABLE.

•• TAKEN FROM FOOTING LAYOUT DRAWING, SHEET 4 OF 34, WASHINGTON STATE HIGHWAY COMMISSION, OLYMPIA, WASHINGTON.

NOTE:
UNLESS OTHERWISE SHOWN, ALL DIMENSIONS ON THIS SHEET ARE IN METERS.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

CENTRAL FERRY HIGHWAY BRIDGE PIERS 3, 4, AND 5
PLAN AND SECTION

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Annex F

Railroad and Highway Embankment Protection Plan

Annex F: Railroad and Highway Embankment Protection Plan

F.1 General

The term "embankments," used in this annex, refers only to the roadway, railroad, and other manmade embankments along the shores of the reservoirs. It does not include the main dam embankments that form part of each dam structure. The modifications described here are based on information available in design memoranda, contract drawings, and aerial photography document the relocations and typical cross-sectional geometry of highway and railroad embankments.

Approximately 140 kilometers (80 miles) of highway and railroad embankment fills exist along the shores of the four lower Snake River reservoirs. They consist of embankments that existed prior to the raising of reservoirs, new embankments for relocated rail and road beds, and existing embankments that were modified or stabilized to function when subject to higher water surface elevations.

Damage to the embankments can occur in two ways. Displacement of the embankment by sliding and settlement occurs as a result of embankment saturation and a subsequent change in water surface elevation. The proposed rapid drawdown of the reservoir water surface initiates this distress. Annex H: Railroad and Roadway Damage Repair Plan, describes the anticipated repair measures for restoring embankments for rail and road service resulting from rapid drawdown. The second major cause of distress is the erosion of embankment materials due to river contact with the embankment. This annex describes the protection measures necessary to protect exposed embankments from the effects of flowing river water.

Currently, most of the embankments have rockfill or riprap erosion protection on the slopes positions several feet above and below the maximum and minimum reservoir elevations. Reservoir drawdown may expose unprotected portions of the embankments during unregulated normal and flood flows, thereby, subjecting these unprotected embankments to erosion. This annex quantifies the amount of additional erosion protection that would be required, based on Corps erosion protection guidelines and assumptions regarding existing embankment geometry and construction.

F.2 Background

A summary of the activities to stabilize, modify, and relocate railroad and roadway embankments prior to raising the four reservoirs may be useful in understanding the rational for selecting the extent of proposed modifications in anticipation of drawdown of the four reservoirs.

F.2.1 McNary Relocations

The McNary Lock and Dam impounds Lake Wallula that results in slackwater extending into the lower Snake River. Ice Harbor Dam is located on the Snake River, 8 miles from its confluence with the Columbia River. The Spokane, Portland, and Seattle Railway (S, P, & S) was located on the north shore of the Snake River. This portion of this rail line connects Pasco, Washington and Spokane, Washington. Because of the raised water surface of the McNary reservoir, a contract to relocate a section of this rail line was performed in 1947 to raise the grade of the line to its current location.

F.2.2 Ice Harbor Relocations

Three rail lines were impacted by raising a reservoir behind Ice Harbor Dam. In addition to the S, P, & S Railroad located on the north shore of the river, the Northern Pacific Railroad spurs from the S, P, & S Railroad at approximately river mile TBD. The Union Pacific Railroad exists on the south shore of the river.

The S, P, & S Railroad between the end of the McNary relocations and Ice Harbor Dam was relocated to raise the grade to pass Ice harbor Dam. Contract 57-127 relocated the S, P, & S Railroad from river mile x at a 0.3% slope to the axis of the dam and at a 0% slope thereafter until the line tied into the existing rail line near Levee Park, river mile TBD. The north shore railroads beyond Levee Park required no major relocation of the rail beds. The raised water surface from the future Ice Harbor reservoir were, in most locations, below the railroad grade. However, because of the significant change in water surface elevation and the effect on embankment stability, portions of the rail beds required extensive stabilization and erosion protection. Some raising of the rail grades was done to provide the necessary freeboard for settlement and wave action protection.

Contract 59-108 provided additional embankment stabilization and erosion protection for the rail section between river mile TBD and TBD. The existing rail beds were used. At Snake River Junction, approximately river mile TBD, the Northern Pacific Railroad parallels the S, P, & S line and often share the same embankment. The Northern Pacific Railroad is located on the riverside of the embankment. Contract 61-139 provided for additional embankment stabilization and erosion protection for the rail section between river mile TBD and TBD. Specific reaches were identified and modifications designed accordingly.

Significant modifications to the Union Pacific Railroad, located on the south shore of the Snake River were necessary to maintain rail service after the pool raise. This rail line provided rail service between Hinkle, Oregon and Spokane, Washington. This Main Line also connects at Ayer, Washington, with the Tekoa-Ayer Branch that in turn connects with the Tucannon Branch and the Camas Prairie Railroad.

Contract 60-117 provided major relocation of the railbed and embankment stabilization of existing segments. Relocation work was done between river miles 12 and 27. Contract 60-142 provided major relocation of the railbed and embankment stabilization of existing segments. Relocation work was done between river miles 27 and 38.

Numerous agreements with the various railroads were made for the removal of abandoned railbeds and the installation of ties and rails. In addition, some sections of the railroad requiring a grade raise, were done by the railroad rather than with a construction contract. Details of some of these agreements and consequent design and construction activities are difficult to find.

F.2.3 Lower Monumental Relocations

The Seattle District Corps of Engineers did portions of the design of Lower Monument relocations. The historical record for this work is not readily available. The S, P, & S Railroad was diverted north at Lower Monumental Dam through Devils Canyon to Kahlotus, Washington. The Northern Pacific rail line along the north shore above Lower Monumental was abandoned. The Union Pacific Railroad on the south shore was relocated. This work included raised and relocated railbeds along the river and new branch lines connecting with the Camas Prairie Railroad and the Tucannon Branch line.

F.2.4 Little Goose Relocations

The branch line connection to the Camas Prairie railroad is located at the Snake River Bridge, river mile TBD. Previous relocations contracts provided the grade raise to river mile TBD. Two major

contracts provided all the railroad relocations for the Little Goose reservoir. Contract 67-104 provides for relocation of the rail beds between river miles TBD and TBD. Contract 68-86 provides for relocation of the rail beds between river miles TBD and TBD. No rail service exists on the south shore of the river.

F.2.5 Lower Granite Relocations

Five major contracts provide all the railroad relocations for the Lower Granite reservoir. Contract 69-19 provides for relocation of the rail beds approaching the downstream of Lower Granite dam and extending through the dam construction zone. This is between river miles TBD and TBD. Contract 73-89 provides for relocation of the rail beds between river miles TBD and TBD. . Contract 73-26 provides for relocation of the rail beds between river miles TBD and TBD. Contract 73-96 provides for relocation of the rail beds between river miles TBD and TBD. Contract 73-102 provides for relocation of the rail beds between river miles TBD and TBD. No rail service exists on the south shore of the river.

F.3 DEVELOPMENT OF METHODOLOGY

The initial approach to quantifying the extent of embankment modification was to perform a field reconnaissance of the embankments. In additional to a visual survey of the embankments, information was collected from aerial survey photographs, U.S. Geological Survey (USGS) 7.5 minute series topographic maps, National Oceanic and Atmospheric Administration (NOAA) navigational maps, and Corps design memoranda and contract drawings. Little information was located on the existence of submerged structures and whether pre-pool structures could be utilized for this drawdown event.

Later interviews with individuals who participated in the contract work for railroad and highway relocations indicated that significant infrastructure existed in each reservoir that may be useful for construction access. Some of pre-reservoir embankments and the temporary construction embankments provide a significant level of erosion protection. Further research confirmed that this was in fact the case and the embankment modification plan was revised.

Aerial photographs of each reservoir were located that clearly showed the pre-reservoir configuration of the Snake River and the on-going construction of railroad relocations. From the photographs, embankment reaches were determined that required protection. The photographs clearly showed where embankments were in contact with the water surface of the river. Most of the photographs were taken when water surface elevations were at minimal levels. Visual determinations of the location of the water surface at 100-year flow levels, 9,056 m^3/s (320,000 cfs) was done and affected embankments identified.

In many cases the reaches requiring embankment protection corresponded to reaches where temporary railroad sections were constructed. These temporary railroad bypasses are termed shooflys. The shooflys provided a temporary bypass so that railroad modifications could proceed on the main line without rail traffic interruptions. In many cases the riverside slopes of the shooflys are protected with riprap or rockfill for flows up to 5,268 m^3/s (186,000 cfs). Long-term stabilization requires the addition of riprap above that flow elevation and the upgrading of rock protection that was designed for only temporary service.

Other segments of new embankment installed during the relocation era serve the purpose of stabilizing the existing embankments. Unfortunately these embankments are reinforced with good riprap in the new reservoir water surface range have only marginal rock protection in the natural river water surface elevations.

The following assumptions and conclusions for embankment protection were established for this study:

- Several assumptions were made in order to attain material quantities. Since specific topographic information is not available and detailed surveys are not practical at this phase, the toe elevation for riprap placement was estimated. The toe was assumed to be located, on average, midway between the elevation of the center river channel and the water surface elevation at flows of 566 m^3/s (20,000 cfs).

- Based on information obtained from design memoranda and contract drawings, most slopes were assumed to be 2.0 horizontal (h):1.0 vertical (v), although there were a few specifically designated as 2.5h:1.0v and 3.0h:1.0v.

- The study team assumed no filter or bedding would be required since most below-reservoir fills are granular or rockfill and do not contain significant fine material to migrate through the riprap.

- No erosion protection is required if the toe of an embankment is located above the natural river (post-drawdown) 100-year flood level (9,056 m^3/s or 320,000 cfs).

- Erosion protection should be provided wherever the embankment toe is located below the natural river (post-drawdown) 100-year flood level (9,056 m^3/s or 320,000 cfs). The erosion protection should extend up from the embankment toe elevation to 1.5 meters (m) (5 feet) above the flood elevation or the crest of the embankment, whichever is less. A diagram illustrating this embankment protection criteria is shown in Figure F1.

- Erosion protection should average in thickness from 8 m to 9 m (27 to 30 inches). Detailed reach water velocities were not determined for this study

Tables F1 to F4 summarize the specific embankment reaches that have been identified to require additional riprap protection. Quantities of riprap for slope protection and surfacing materials for vehicle access are shown in the tables. Figure F2 show several proposed modifications utilized for modification of existing embankments.

Table F1. Ice Harbor Reservoir Embankment Modification Reaches

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
N1	S, P, & S RR	11	2950	27,213	983
N2	S, P, & S RR	25	700	4,348	233
N3	S, P, & S RR	24	1100	7,516	367
N4	S, P, & S RR	24	1300	8,882	433
N5	S, P, & S RR	21	2000	16,978	667
N6	S, P, & S RR	19	500	3,209	167
N7	S, P, & S RR	14	700	5,073	233
N8	NPRR	27	1750	13,044	583
N9	NPRR	28	4075	27,842	1,358
S1	UPRR	14	11740	0	3,913
S2a	UPRR	15	400	1,926	133
S2b	UPRR	15	439	2,113	146
S3	UPRR	16	5232.57	25,188	1,744
S4	UPRR	23	7624.3	39,069	2,541
S5	UPRR	26	1874.2	11,059	625
S6	UPRR	29	2500	1,941	833
S7	UPRR	32	6119.2	32,307	2,040
S8	UPRR	34	4700	16,056	1,567
			55,704	**243,763**	**18,568**

Table F2. Lower Monumental Reservoir Embankment Modification Reaches

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
S1	UPRR	43	6,180	26,306	2,060
S2a	UPRR	48	7,520	32,009	2,507
S2b	UPRR	50	5,560	24,363	1,853
S3	UPRR	52	1,750	8,763	583
S4	UPRR	56	7,443	34,477	2,481
S5	UPRR	63	4,900	20,857	1,633
S6	County Road	66	9,680	22,621	3,227
S7	County Road	70	10,920	35,545	3,640

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
N1	CPRR	65	4,940	20,409	1,647
N2	CPRR	67	3,190	13,578	1,063
			62,083	**238,928**	**20,694**

Table F3. Little Goose Reservoir Embankment Modification Reaches

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
N1	CPRR	72	4,900	14,723	1,633
N2	CPRR	74	11,000	36,723	3,667
N3	CPRR	78	2,847	9,267	949
N4	CPRR	80	7,576	33,196	2,525
N5	CPRR	89	11,496	50,373	3,832
N6	CPRR	93	6,086	25,905	2,029
N7	CPRR	99	4,086	20,973	1,362
N8	CPRR	103	7,521	34,838	2,507
N9	CPRR	105	7,277	31,886	2,426
S1	Central Ferry				
S2	County Road				0
			62,789	**257,884**	**20,930**

Table F4. Lower Granite Reservoir Embankment Modification Reaches

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
N1	CPRR	109	17,180	77,429	5,727
N2	CPRR	113	5,000	15,440	1,667
N3	CPRR	117	8,567	35,393	2,856
N4	CPRR	120	4,878	24,428	1,626
N5	CPRR	123	10,026	56,483	3,342
N6	CPRR	124	1,593	8,974	531
N7	CPRR	127	12,765	65,522	4,255
N8	CPRR	133	9,493	51,104	3,164

Reach Number	Railroad Highway Designation	Approximate Location (River Mile)	Reach Length (Feet)	Riprap Volume (cy)	Base Volume (cy)
N9	CPRR	138	6,600	17,627	2,200
S1	Highway 12	135	18,600	51,229	
LL1	N Lewiston Levee				
LL2	W Lewiston Levee				
			57,502	**352,400**	**25,367**

F.4 Construction Scenario

Protection of the existing embankment structures on the lower Snake River reservoirs provide some logistical challenges in preparing design approaches and staging construction operations. The embankments to be protected cover some 156 kilometers (97 miles) of reservoir shoreline, offer difficult land access, and require placement of bands of riprap extending along steep slopes. The major construction operations are:

1. Quarry Development and Production
2. Stockpiling of Riprap and other Materials
3. Development and Upgrading of Access Roads to the River
4. Treatment and modifications to river access roads
5. Placement of riprap
6. Placement of drainage modifications.

F.4.1 Quarry Development and Production

The drawdown of the four lower Snake River reservoirs would require approximately 750,000 m^3 (1 million cy) of riprap material for protection of embankments, drainage modifications, and bridge abutments. The size and gradation requirements for this material, ranging from (1-foot) to 0.8-m (2.5-foot), is generally available from existing quarry sites in the Lower Granite reservoir and other potential sites in the Ice Harbor and Lower Monumental reservoirs. Based on past experience and the general practice of quarry development within the Lower Granite reservoir area, about two and one-half times as much material must be processed as can be used for riprap. If 0.75 million m^3 (1 million cy) is to be used, about 1.8 million cubic meters (2.5 million cubic yards) will have to be processed.

Identification of specific quarry sources requires extensive explorations to ascertain rock quality and quantity. Without those explorations, the study team assumed general locations for major quarries in the Ice Harbor and Lower Monumental reservoirs and utilized previously developed quarries for the Lower Granite relocations.

Since dam breaching will be done over a period of 2 consecutive construction seasons, not all quarries need to be developed at the same time. Embankments in the Lower Granite and Little Goose reservoirs are scheduled to be modified during the first breach season. This requires that the 5-6 land-based quarries along the Lower Granite reservoir be developed and sufficient material stockpiled at each quarry for use following drawdown. In addition, the designated quarry near the confluence with the Palouse River in the Lower Monumental reservoir must be developed and riprap loaded, barged, and stockpiled at 6 stockpile locations in the reservoir. These underwater locations will later become accessible when the water

surface is drawdown. Overland transportation of rock materials to all the necessary reaches would be a monumental task of developing haul roads and hauling great distances.

In preparation for the following breach season at Lower Monumental and Ice Harbor dams, an additional quarry must be developed in the Ice Harbor reservoir. Riprap from this quarry and the Lower Monumental quarry will be loaded barged, and stockpiled at 6 stockpile locations in the reservoir. These underwater locations will later become accessible when the water surface is drawdown. Overland transportation of rock materials to all the necessary reaches would be a monumental task of developing haul roads and hauling great distances.

Use of a commercial source of riprap was assumed for required stabilization of levee sections in the vicinity of Lewiston, Idaho.

Steps in quarry development and material processing are generally as follows:

1) Establish access and haul roads.
2) Establish stockpile and disposal areas (these will change throughout the life of the quarry).
3) Establish loading, weighing, and traffic control facilities (usually one-time set-up for the life of the quarry).
4) Strip off and dispose of overburden from the usable rock formation (infrequent cycles).
5) Drill and shoot rock into manageable size material (periodic cycles).
6) Load and haul shotrock to grisly and bar screens (continuous process).
7) Crush oversize material, drill and shoot boulders, send to grisly and bar screens (periodic cycles).
8) Stockpile according to material size (continuous).
9) Load and haul finished product (frequent cycles or continuous).

Overland transportation of riprap from the quarries along the Lower Granite reservoir is a standard operation similar is scope to the pre-reservoir relocations contract work. Transporting the large volumes of material needed for the other three reservoirs in one or two construction seasons would provide a severe strain on the existing infrastructure and road system around the lower Snake River reservoirs. This factor, plus the very limited access to placement sites along the reservoirs, forces the more practical and economical choice of barging the riprap to placement locations.

Other factors that influence the overall duration of reservoir construction activities are productivity rates for quarrying rock, barge loading, barge transport, barge unloading, stockpiling, and placement of materials. For the development of unit prices, productivity rates for these activities were assumed based on the selection of a piece of "prime equipment" around which a crew of support equipment and labor were assembled. Typical construction activities, prime equipment, and corresponding productivity rates for riprap construction were determined and are summarized in Table F5.

When the supply requirements exceed the average productivity of a "prime equipment" set-up, then additional crew set-ups can be added, as long as working space is sufficient. Based on these assumptions and the use of multiple crew set-ups and multiple sources of suitable riprap, coupled with the already existing water shipping capabilities of the four lower Snake River reservoirs, the riprap supply needs can be met in one construction season and the riprap placement completed in one season.

Table F5. Typical Riprap Productivity Rates

Activity	Prime Equipment	Average Productivity
Quarrying Rock	RT Loader with 10.7 m^3 Bucket	612 m^3/hr
Barge Loading	RT Loader with 10.7 m^3 Bucket	612 m^3/hr
Barge Transport	Barge with 1,200-hp Tug (1,150 m^3/barge)	16 km/hr
Riprap Placement	CAT 235D Excavator with 1.6 m^3 Bucket	30 m^3/hr

F.4.2 Stockpiling of Riprap and other Materials

Because of the poor overland access to many of the placement sites below Lower Granite Dam, a logical approach is to utilize barge transportation of riprap to those sites. For purposes of developing a complete plan and realistic estimate, in-water stockpile sites conveniently located to placement sites have been determined. These sites are situated such that after drawdown, they are above the low water elevation and allow vehicle loading operations to be staged at the stockpile site. The location of these sites has been coordinated with projected locations of spawning and rearing areas in the natural river to minimize impacts to those favorable areas. Since stockpile locations must be above the low water elevation and the work will done soon after drawdown before sediment transport in the river system has stabilized, the impacts resulting from in-water stockpiling should be minimal.

Prior to the respective reservoir drawdown, riprap and base materials are to be stockpiled. This work should be scheduled to be completed prior to the spring runoff in approximately March of the year that drawdown occurs.

F.4.3 Development and Upgrading of Access Roads to the River

Overland vehicle access to the Snake River can be extremely difficult in some locations. County and state roads parallel the river is some locations, there are a number of roads that access farms, grain elevators, and recreational areas. A number of the existing roads will require improvements and more frequent maintenance to provide access for large construction equipment. While the bulk of the materials will be pre-placed by barge, other materials, equipment, and personnel must access the work areas via these existing roads. Upgrades include regrading, resurfacing, widening at selected points, and adding turnouts. Maintenance activities include continued grading and pothole repair, dust control, and rock additions to maintain all-weather access.

Access to the Lower Granite reservoir will be primarily from Highway 193, a Whitman Country Road. This roadway parallels the reservoir from near Lewiston to Wawawai, within 3 miles of Lower Granite Dam. Access below Wawawai will be attained along the inundated road and railbeds from the either end.

Major roadways to the Little Goose reservoir include Highway 12 at Central Ferry and Highway TBD at Almota. Several minor roads provide intermediate access to the Little Goose reservoir. These minor roads will require modifications as described above.

Major roadways to the Lower Monumental reservoir include Highway 261 at Lyons Ferry and the Little Goose project access road just upstream. Several minor roads provide intermediate access to the Lower Monumental reservoir. These minor roads will require modifications as described above.

Access to the Ice Harbor reservoir is possible via a number of existing county and private roadways. Agricultural development is greatest in the lower 15 miles of this reservoir and access along this reach is available.

F.4.4 Treatment and modifications to river access roads

Modifications to the local infrastructure described above provides access to the river at numerous locations. Further access is necessary to travel along the river to all the reaches the require embankment stabilization and modifications to drainage structures. Access will be provided on existing, currently inundated, construction access roads, abandoned rail and road beds. Many of these road and rail beds were not removed after construction of the new rail and road sections. These will once again be available for use after drawdown.

Several treatments will be necessary to make these roadways serviceable. Many will require grading to remove sediments that have accumulated on the surfaces. The conditions of the aggregate surfacing on these road sections is not known. It is assumed that many of these sections will require the addition of material. Sections must be widened, turnouts added, and other modifications for construction operations.

The major problem with sequencing the work as a post-drawdown operation is the slow draining of the inundated embankments. Drawdown occurs between the months of August and December. Obviously, vehicle access to the old roadways is not possible until the materials "dry out." Spring runoff between March and July prevents in-water placement of riprap. Work on the placement of riprap will not begin until approximately August. This allows 9-12 months for the embankments to drain and allow vehicle access to the river.

F.4.5 Placement of Riprap

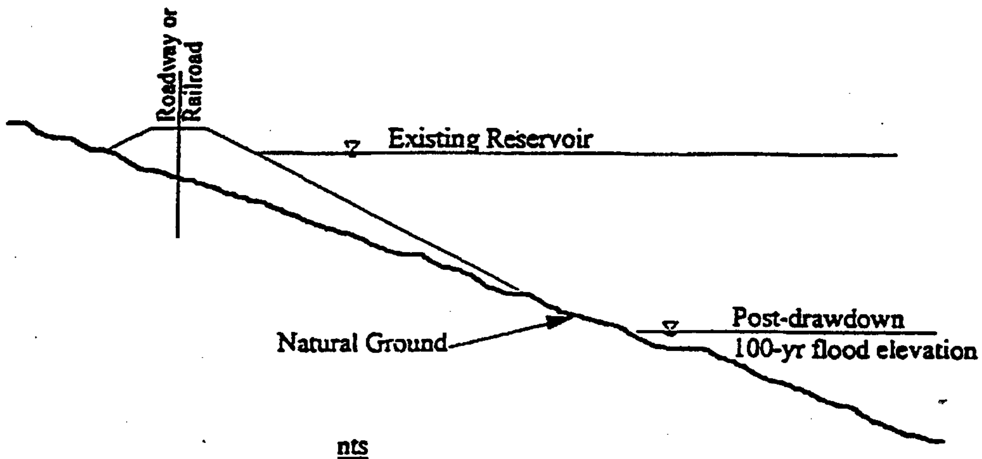
Riprap placement is a tedious and time-consuming operation. It generally requires the use of an excavator to place the rock and key the rock mat together. Areas that are difficult to access will require a dozer to pioneer a road to the placement area so the excavator can reach the placement area and haul vehicles can deposit rock near the excavator. Bedding material under the riprap will not be necessary for most of the reaches since the underlying material is rockfill, gravel materials, or a smaller riprap. The riprap must be keyed into the river channel to prevent undermining of the rock when high velocity flows occur. Because of the slow production rate for riprap placement, work must occur at many placement sites concurrently to get the work done at the projected schedule.

F.4.6 Placement of drainage modifications

A detailed discussion of drainage system modifications is presented in Annex G – Drainage Structures Modifications.

Embankment Protection Criteria

1. No Protection Required: Post-drawdown 100-yr flood (320,000 cfs, or 9,056 cms) elevation below toe of embankment.



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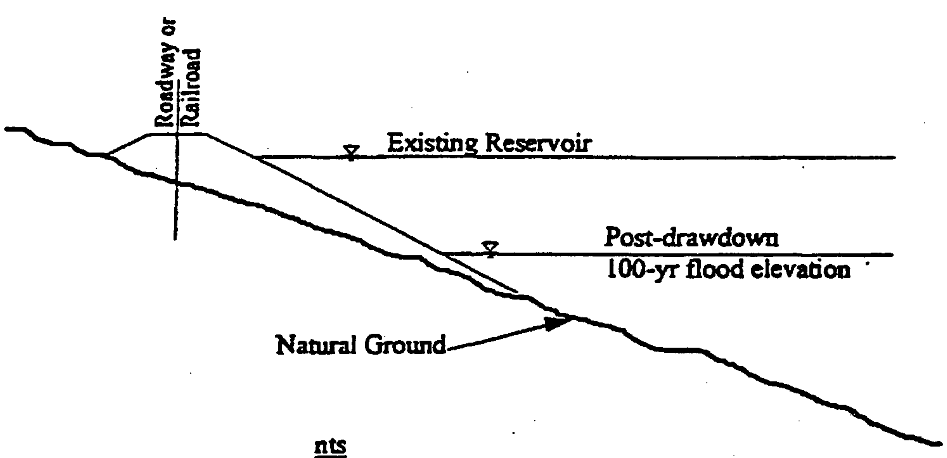
2. Erosion Protection Required: Post-drawdown 100-yr flood (320,000 cfs, or 9,056 cms) elevation above toe of embankment.



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US Army Corps of Engineers Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
EMBANKMENT PROTECTION CRITERIA

Figure: F1

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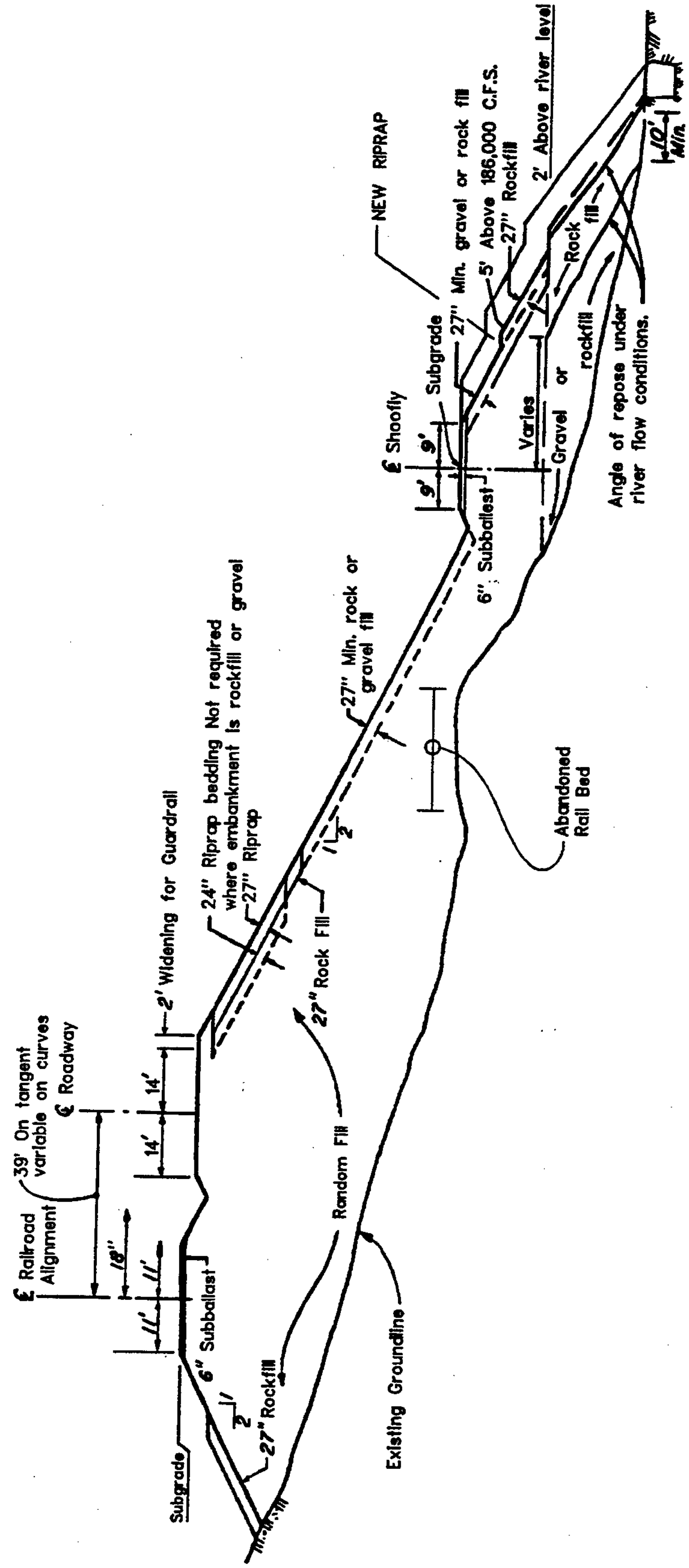

TYPICAL COMBINED EMBANKMENT SECTION

NOTES:

1. SHOOFLY IS A TEMPORARY RAILROAD EMBANKMENT TO DIVERT RAIL TRAFFIC DURING CONSTRUCTION OF NEW RAIL BEDS.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

EMBANKMENT PROTECTION DETAILS

Figure: F2

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SHEET MAIN SCALE:

Annex G

Drainage Structures Protection Plan

Annex G: Drainage Structures Protection Plan

G.1 General

Numerous drainage structures exist along the reservoirs of Ice Harbor, Lower Monumental, Little Goose, and Lower Granite dams. These structures were designed to allow passage of water from existing upslope drainages through highway and railroad embankments into the reservoirs created by the dams. They range in size from ranging in size from 200-millimeter (mm) (8-inch) diameter PVC pipes to 6.1- by 3.0-meter (m) (20- by 10-foot) corrugated metal arch pipes.

A review of as-constructed contract drawings for various projects indicates additional culverts are present below current reservoir levels. These structures were installed for the original railroad construction as well as for temporary modification of the railroad alignment during dam construction. These culverts are typically 900 mm (36 inches) or larger in diameter. With reservoir impoundment and relocation of post-reservoir highway and railroad embankments, additional ponded areas were created on the landward side of relocated highway and railroad embankments, thus requiring additional post-reservoir drainage.

The existing drainage structures at or above the reservoir pools were identified, located, and cataloged in August of 1995 by visual reconnaissance from a boat along the shore of the four reservoirs. The locations of these drainage structures and the inventory survey data for each of the reservoirs is presented in Plates 6-1 through 6-8 of a separate report titled *Lower Snake River Reservoir Stabilization Plan* (Raytheon 1997). The majority of the structures are 300- to 1500-mm (12- to 60-inch) diameter corrugated metal pipe (CMP) culverts with the remaining structures consisting of concrete or CMP arch culverts with cast-in-place headwalls and box culverts. At some locations, the culverts have a grouted rock apron at the outlet.

G.2 Standard Modifications

Seepage and surface runoff that flows down slope into natural depressions and impoundments created by man-made fills tends to locally saturate the soil and can cause instability. To control seepage from these depressions and fills, transverse interceptor drains or culverts were constructed in conjunction with the construction of highway and railroad embankments.

The drainage structures visible along the reservoir slopes encompass a range of elevations above the existing water surface depending on former and relocated alignments. Tables G1 through G4 summarize information on the physical characteristics of these drainage structures and the proposed modifications. This information, in conjunction with the review of the construction documents, was used to plan the drainage structure modifications for the proposed drawdown of the lower Snake River reservoirs.

Table G1. Lower Granite Drainage Structures Treatment

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)
D-1	36	Inch	CMP	3	ED	130.80	R	D-30	24	Inch	CMP	3	ED	120.80	R	D-59	36	Inch	CMP	4	ED	113.50	R
D-2	30	Inch	CMP	3	ED	130.55	R	D-31	2@48	Inch	CMP	3	ED	120.50	R	D-60	36	Inch	CMP	3	ED	113.20	R
D-3	36	Inch	CMP	3	SP	128.60	R	D-32	24	Inch	CMP	3	ED	120.20	R	D-61	24	Inch	CMP	4	ED	113.10	R
D-4	30	Inch	CMP	4	SP	128.45	R	D-33	36	Inch	CMP	3	ED	120.00	R	D-62	36	Inch	CMP	4	ED	112.90	R
D-5	24	Inch	CMP	2	SP	127.70	R	D-34	18	Inch	CMP	3	ED	119.80	R	D-63	30	Inch	CMP	5	ED	112.40	R
D-6	30	Inch	CMP	3	ED	127.45	R	D-35	10	Inch	CMP	6	ED	119.55	R	D-64	24	Inch	CMP	4	ED	112.00	R
D-7	2@36	Inch	CMP	3	ED	126.95	R	D-36	10	Inch	CMP	5	ED	119.40	R	D-65	24	Inch	CMP	3	ED	111.85	R
D-8	24	Inch	CMP	4	ED	126.70	R	D-37	30	Inch	CMP	2	ED	119.30	R	D-66	24	Inch	CMP	4	ED	111.80	R
D-9	36	Inch	CMP	3	ED	126.30	R	D-38	10	Inch	CMP	4	ED	119.05	R	D-67	30	Inch	CMP	4	ED	111.40	R
D-10	36	Inch	CMP	3	ED	126.00	R	D-39	24	Inch	CMP	3	ED	118.80	R	D-68	12	Inch	CMP	8	ED	110.90	R
D-11	30	Inch	CMP	2	ED	124.95	R	D-40	30	Inch	CMP	3	ED	118.45	R		18	Inch	CMP	0.5	ED	110.90	R
D-12	24	Inch	CMP	3	ED	124.50	R	D-41	24	Inch	CMP	2	ED	118.25	R		18	Inch	CMP	0.5	ED	110.90	R
D-13	2@48	Inch	CMP	4	ED	124.20	R	D-42	24	Inch	CMP	4	ED	117.95	R	D-69	48	Inch	CMP	4	ED	110.25	R
D-14	12	Inch	CMP	6	ED	123.85	R	D-43	30	Inch	CMP	4	ED	117.60	R	D-70	36	Inch	CMP	3	ED	110.00	R
D-15	24	Inch	CMP	3	ED	123.80	R	D-44	5	Feet	CMP	3	ED	117.30	R	D-71	3@30	Inch	CMP	3	SP	109.90	R
D-16	5	Feet	CMP	3	ED	123.80	R	D-45	24	Inch	CMP	3	ED	117.15	R	D-72	36	Inch	CMP	2	SP	109.80	R
D-17	2@48	Inch	CMP	3	ED	123.70	R	D-46	5	Feet	CMP	3	ED	117.00	R	D-73	48	Inch	CMP	2	SP	109.50	R
D-18	36	Inch	CMP	5	ED	123.60	R	D-47	36	Inch	CMP	3	ED	116.60	R	D-74	24	Inch	CMP	3	SP	108.95	R
D-19	18	Inch	CMP	4	ED	123.60	R	D-48	30	Inch	CMP	4	SP	116.40	R	D-75	30	Inch	CMP	3	SP	108.85	R
D-20	18	Inch	CMP	4	ED	123.45	R	D-49	12	Inch	CMP	8	SP	116.30	R	D-76	2@48	Inch	CMP	4	ED	108.70	R
D-21	10	Inch	CMP	8	SP	122.80	R	D-50	24	Inch	CMP	4	SP	116.10	R	D-77	36	Inch	CMP	3	ED	108.30	R
D-22	24	Inch	CMP	3	SP	122.75	R	D-51	30	Inch	CMP	4	SP	116.00	R	D-78	48	Inch	CMP	3	SP	108.00	R
D-23	24	Inch	CMP	3	SP	122.50	R	D-52	36	Inch	CMP	3	SP	115.65	R	D-79	18	Inch	CMP	4	SP	108.15	R
D-24	24	Inch	CMP	3	SP	122.20	R	D-53	36	Inch	CMP	3	ED	115.00	R	D-80	48	Inch	CMP	2	SP	108.20	L
D-25	30	Inch	CMP	3	SP	122.10	R	D-54	30	Inch	CMP	3	ED	114.55	R	D-81	18	Inch	CMP	3	ED	131.50	L
D-26	24	Inch	CMP	3	ED	121.80	R	D-55	48	Inch	CMP	4	ED	114.40	R	D-82	6	Feet	CMP	3	ED	137.30	R
D-27	24	Inch	CMP	3	ED	121.60	R	D-56	24	Inch	CMP	4	ED	114.25	R	D-83	6	Feet	CMP	4	ED	133.40	R
D-28	24	Inch	CMP	3	ED	121.30	R	D-57	36	Inch	CMP	3	ED	113.90	R	D-84	48	Inch	CMP	3	ED	132.90	R
D-29	36	Inch	CMP	3	ED	121.00	R	D-58	36	Inch	CMP	3	ED	113.80	R	D-85	2@24	Inch	CMP	4	ED	132.80	R

Treatment Codes: ED =Energy Dissipater; SP = Slope Protection; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CMPA = Corrugated Metal Pipe Arch

Table G1 con't. Lower Granite Drainage Structures Treatment

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)
D-86	2@36	Inch	CMP	3	ED	132.55	R	D-113	24	Inch	CMP	4	ED	142.80	L	D-137	24	Inch	CMP	3	SP	138.40	R
D-87	36	Inch	CMP	3	ED	132.30	R	D-114	24	Inch	CMP	10	ED,CLC	142.95	L	D-138	48	Inch	CMP	2	ED	137.95	R
D-88	24	Inch	CMP	3	ED	132.15	R	D-115	2@5	Feet	CMP/ G	8	EDC	143.60	L	D-139	18	Inch	CMP	2	SP	137.50	R
D-89	24	Inch	CMP	3	ED	132.10	R	D-116	36	Inch	CMP	0.5	ED	144.30	L								
D-90	24	Inch	CMP	3	ED	132.00	R	D-117	12	Inch	CMP	15	ED,CLC	144.80	L								
D-91	36	Inch	CMP	3	ED	131.80	R	D-118	24	Inch	CMP	20	ED,CLC	142.50	R								
D-92	24	Inch	CMP	4	ED	131.35	R	D-119	12	Inch	CMP	12	ED,CLC	142.40	R								
D-93	18	Inch	CMP	-1	ED	131.00	L	D-120	12	Inch	CMP	12	ED,CLC	142.30	R								
D-94	30	Inch	CMP	-1.5	ED	131.20	L	D-121	12	Inch	CMP	12	ED,CLC	142.10	R								
D-95	18	Inch	CMP	3	ED	131.30	L	D-122	12	Inch	CMP	10	ED,CLC	142.00	R								
D-96	24	Inch	CMP	4	ED	133.50	L	D-123	12	Inch	CMP	10	ED,CLC	141.85	R								
D-97	18	Inch	CMP	3	ED	133.85	L	D-124	12	Inch	CMP	12	ED,CLC	142.75	R								
D-98	18	Inch	CMP	3	ED	134.40	L		12	Inch	CMP	30	ED,CLC	142.8	R								
D-99	18	Inch	CMP	3	ED	134.60	L	D-125	12	Inch	CMP	8	ED	141.60	R								
D-100	18	Inch	CMP	4	ED	134.70	L	D-126	42	Inch	CMP	4	ED	141.45	R								
D-101	2@48	Inch	CMP	1	EDC	134.90	L	D-127	30	Inch	CMP	2	ED	141.30	R								
D-102	3@5	Feet	CMP	0	EDC	135.30	L	D-128	18	Inch	CMP	13	ED,CLC	141.10	R								
D-103	18	Inch	CMP	2	ED	135.50	L	D-129	36	Inch	CMP	4	ED	140.90	R								
D-104	18	Inch	CMP	3	ED	135.80	L		18	Inch	CMP	0	ED	140.90	R								
D-105	18	Inch	CMP	3	ED	136.10	L	D-130	24	Inch	CMP	5	ED	140.80	R								
D-106	18	Inch	CMP	3	ED	136.25	L	D-131*	24	Inch	Conc	3	ED	3.80	R								
D-107	24	Inch	CMP	3	ED	136.70	L	D-132*	24	Inch	CMP	15	ED,CLC	0.40	R								
D-108	18	Inch	CMP	3	ED	137.00	L		12	Inch	CMP	15	ED,CLC	0.40	R								
D-109	36	Inch	CMP	-1	EDC	138.10	L	D-133*	6	Feet	CMP	3	ED	0.20	R								
D-110	36	Inch	CMP	20	ED,CLC	138.20	L	D-134	4	Feet	CMP	2	ED	138.75	R								
D-111	24	Inch	CMP	10	ED,CLC	141.60	L	D-135	24	Inch	CMP	2	ED	138.60	R								
D-112	12	Inch	CMP	5	ED	141.55	L	D-136	4	Feet	CMP	2	ED	138.50	R								

Treatment Codes: ED =Energy Dissipater; SP = Slope Protection; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CMPA = Corrugated Metal Pipe Arch

*** Indicates drainage structure on Clearwater River**

Table G2. Ice Harbor Drainage Structures Treatment

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)
D-1	8	Inch	PVC	4	ED	11.90	R	D-30	8	Feet	CHA	15	EDC,CLC	31.20	R	D-55	12	Inch	CMP	10	ED,CLC	40.20	R
D-2	36	Inch	CMP	5	SP	13.61	R	D-31	24	Inch	CMP	40	SP	32.20	R	D-56	18	Inch	CMP	15	ED,CLC	41.50	R
D-3	36	Inch	CMP	5	SP	14.00	R	D-31	18	Inch	Steel	15	ED	32.20	R	D-57	5	Feet	CHA	25	ED,CLC	40.50	L
D-4	5	Feet	CMP	3	ED	14.80	R	D-32	24	Inch	CMP	40	SP	32.30	R	D-58	24	Inch	CMP	10	ED,CLC	38.25	L
D-5	36	Inch	CMP	6	SP	18.00	R	D-32	24	Inch	Steel	15	ED	32.30	R	D-59	24	Inch	CMP	10	ED,CLC	37.50	L
D-6	8x8	Feet	CBC	5	EDC	18.20	R	D-33	2	Feet	CMP	2	ED	32.60	R	D-60	24	Inch	CMP	3	ED	37.40	L
D-7	36	Inch	CMP	2	ED	18.40	R	D-33	3	Feet	CMP	3	ED	32.60	R	D-61	24	Inch	CMP	3	ED	37.30	L
D-8	36	Inch	CMP	0	ED	18.60	R	D-34	24	Inch	CMP	50	SP,CLC	32.70	R	D-62	5	Feet	CMPA	5	ED	37.40	L
D-9	36	Inch	CMP	0	ED	19.10	R	D-35	30	Inch	WS	30	SP,CLC	34.65	R	D-63	30	Inch	CMP	10	ED,CLC	37.20	L
D-10	4.5	Feet	CMP	10	SP,CLC	21.20	R	D-36	8	Feet	CHA	30	SPC,CLC	35.15	R	D-64	12	Inch	CMP	3	ED	37.10	L
D-11	3	Feet	CMP	?	SP	21.60	R	D-37	5	Feet	CMP	6	ED	35.94	R	D-65	18	Inch	CMP	6	ED	37.05	L
D-12	18	Inch	CMP	2	SP	21.57	R	D-38	18	Inch	CMP	2	ED	36.10	L	D-66	4	Feet	CMP	10	ED,CLC	28.70	L
D-13	36	Inch	CMP	15	SP,CLC	21.60	R	D-39	12	Inch	CMP?	3	ED	36.00	L	D-67	24	Inch	CMP	25	ED,CLC	28.00	L
D-14	36	Inch	CMP	20	SP,CLC	21.70	R	D-40	12	Inch	CMP	15	ED,CLC	35.73	L	D-68	24	Inch	CMP	20	SP,CLC	26.40	L
D-15	36	Inch	CMP	?	SP	21.75	R	D-41	24	Inch	CMP	6	ED	35.40	L	D-69	3	Feet	CMP	30	SP,CLC	26.35	L
D-16	Unk		CMP	<5	SP	21.85	R	D-42	24	Inch	CMP	1	ED	35.05	L	D-70	6	Feet	CMP	30	ED,CLC	24.40	L
D-17	4.5	Feet	CMP	25	SP,CLC	22.00	R	D-43	36	Inch	CMP	3	ED	34.15	L	D-71	3	Feet	CMP	30	SP,CLC	23.90	L
D-18	4	Feet	CMP ?	-1.5	ED	22.15	R	D-44	Unk	Inch	Unk	Unk	ED	33.65	L	D-72	3	Feet	CMP	30	SP,CLC	23.85	L
D-19	4	Feet	CMP	0	ED	22.70	R	D-45	18	Inch	CMP	5	ED	31.30	L	D-73	24	Inch	CMP	40	SP,CLC,	23.00	L
D-20	18	Inch	Metal	10	ED,CLC	25.45	R	D-46	15	Feet	CMPA	-3	ED	30.40	L	D-74	3	Feet	CMP	40	SP,CLC	22.60	L
D-21	2@ 6	Feet	CMP	-3/-1.5	EDC	26.20	R	D-47	24	Inch	CMP	4	ED	30.17	L	D-75	24	Inch	CMP	30	SP,CLC	17.40	L
D-22	18	Inch	ABS	25	ED,CLC	26.50	R	D-48	15	Feet	CMPA	-3	EDC	29.80	L	D-76	30	Inch	CMP	5	ED	17.10	L
D-23	24	Inch	CMP	5	ED	26.83	R	D-49	8	Feet	CHA	12	EDC	37.70	R	D-77	20x10	Feet	CMPA	-1	ED	17.00	L
D-24	4	Feet	CMP	35	ED,CLC	27.60	R	D-50	24	Inch	CMP	-0.5	ED	38.40	R	D-78	4	Feet	CMP	10	SP,CLC	16.80	L
D-25	4	Feet	CMP	6	ED	28.00	R	D-51	12	Inch	CMP	10	ED,CLC	39.15	R	D-79	<12	Inch	CMP	6	SP	15.90	L
D-26	3	Feet	CMP	35	ED,CLC	29.40	R	D-51	12	Inch	CMP	3	ED	39.15	R	D-80	24	Inch	CMP	15	SP,CLC	14.10	L
D-27	18	Inch	CMP	20	SP,CLC	30.50	R	D-52	12	Inch	CMP	7	ED	39.30	R	D-81	4	Feet	CMP	30	SP,CLC	13.00	L
D-28	18	Inch	ABS	30	ED,CLC	30.80	R	D-53	12	Inch	CMP	6	ED	39.50	R	D-82	24	Inch	CMP	40	SP,CLC	10.50	L
D-29	18	Inch	ABS	30	ED,CLC	31.00	R	D-54	12	Inch	CMP	8	ED	39.80	R	D-82	24	Inch	CMP	25	SP,CLC	10.50	L

Treatment Codes: SP = Slope Protection; ED =Energy Dissipater; SPC = Slope Protection with concrete treatment; EDC = Energy Dissipater with concrete treatment; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CBC = Concrete Box Culvert; CMPA = Corrugated Metal Pipe Arch

Table G3. Lower Monumental Drainage Structures Treatment page 1

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River - mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)
D-1	30	Inch	CMP	5	ED	61.15	R	D-23	30	Inch	CMP	30	ED,CLC	66.50	R		Includes Concrete Headwall						
D-2	42	Inch	CMP/G	40	EDC, CLC	62.80	R	D-24	30	Inch	CMP	45	ED,CLC	66.60	R	D-44	3	Feet	CMP	30	SP,CLC	58.70	L
D-3	36	Inch	CMP	40	ED,CLC	63.00	R	D-25	30	Inch	CMP	50	ED,CLC	66.70	R	D-45	12'x 10'	Arch	Conc	3	EDC	56.05	L
D-4	36	Inch	CMP	30	ED,CLC	63.10	R	D-26	30	Inch	CMP	50	ED,CLC	66.80	R		Includes large Concrete Headwall						
D-5	30	Inch	CMP	35	SP,CLC	63.20	R	D-27	30	Inch	CMP	50	ED,CLC	66.90	R	D-46	3	Feet	CMP/G	5	ED	54.80	L
D-6	30	Inch	CMP	10	SP,CLC	63.30	R	D-28	8	Feet	CMP	70	SPC	69.90	R	D-47	30	Inch	CMP	10	ED,CLC	53.10	L
D-7	30	Inch	CMP/G	30	SPC, CLC	63.40	R	D-29	48	Inch	CMP	120	SPC	70.20	R	D-48	8	Feet	CMP/G	1	EDC	51.90	L
D-8	30	Inch	CMP	40	SP,CLC	63.50	R	D-30	36	Inch	CMP	12	ED	69.10	L		Conc Grouted Riprap around Culvert						
D-9	24	Inch	CMP	40	SP,CLC	63.60	R	D-31	5	Feet	CMP	1	ED	68.50	L	D-49	30	Inch	CMP	15	ED,CLC	51.60	L
D-10	30?	Inch	CMP	40	SP,CLC	63.80	R	D-32	48	Inch	CMP	0	ED	68.30	L	D-50	24	Inch	CMP	4	ED	51.20	L
D-11	30	Inch	CMP	35	SP,CLC	63.90	R	D-33	24	Inch	CMP	8	ED	67.10	L	D-51	12'x 6'	Conc Box Culvert		Below	EDC	50.90	L
D-12	30?	Inch	CMP	35	SP,CLC	64.00	R	D-34	5	Feet	CMP	50	SP,CLC	67.10	L		Includes large Concrete Headwall						
D-13	30	Inch	CMP	40	SP,CLC	64.20	R	D-35	4	Feet	CMP	15	ED	66.00	L	D-52	4	Feet	CMP/G	1	EDC	50.30	L
D-14	8	Feet	CMP/G	4	EDC	64.50	R	D-36	36	Inch	CMP	3	ED	65.70	L	D-53	18	Inch	CMP	15	SP,CLC	50.20	L
D-15	30	Inch	CMP	40	ED	64.73	R	D-37	12	Inch	CMP	2	ED	65.55	L	D-54	18	Inch	CMP	15	SP,CLC	50.15	L
D-16	30	Inch	CMP	50	ED,CLC	64.80	R	D-38	36	Inch	CMP	4	ED	65.30	L	D-55	3	Feet	CMP	1	SPC	50.00	L
D-17	Unknown			Below	ED	65.10	R	D-39	24	Inch	CMP	3	ED	65.10	L								
D-18	30	Inch	CMP	40	ED,CLC	65.40	R	D-40	36	Inch	CMP	3	ED	64.05	L								
D-19	54	Inch	CMP/G	50	EDC, CLC	65.45	R	D-41	8	Feet	CMP	4	EDC	59.25	L								
D-20	30	Inch	CMP/G	40	EDC,CLC	65.80	R	D-42	24	Inch	CMP	50	SP,CLC	59.25	L								
D-21	30	Inch	CMP	60	ED,CLC	65.90	R	D-43	36	Inch	CMP	60	SPC,CLC	59.00	L								
D-22	36	Inch	CMP	50	ED, CLC	66.30	R																

Treatment Codes: SP = Slope Protection; ED =Energy Dissipater; SPC = Slope Protection with concrete treatment; EDC = Energy Dissipater with concrete treatment; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CMPA = Corrugated Metal Pipe Arch; CMP/G = Corrugated Metal Pipe with grouted rock apron

Table G3 con't. Lower Monumental Drainage Structures Treatment page 2

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L /R)
	Conc Grouted Riprap around Culvert								Conc Grouted Riprap around Culvert														
D-56	18	Inch	CMP	15	SP,CLC	49.90	L	D-76	4	Feet	CMP/G	5	EDC	44.60	L								
D-57	18	Inch	CMP	15	SP,CLC	49.80	L	D-77	2	Feet	CMP/G	6	EDC	44.50	L								
D-58	24	Inch	CMP	15	SP,CLC	49.70	L		Conc Grouted Riprap around Culvert														
D-59	24	Inch	CMP	15	SP,CLC	49.60	L	D-78	Unknown (Submerged ?)				EDC	44.15	L								
D-60	3	Feet	CMP/G	3	EDC	49.40	L	D-79	4	Feet	CMP/G	3	EDC	43.60	L								
D-61	18	Inch	CMP	8	ED	49.20	L	D-80	18	Inch	CMP	10	SPC	43.40	L								
D-62	18	Inch	CMP	8	ED	49.10	L	D-81	18	Inch	CMP	12	ED,CLC	43.00	L								
D-63	24	Inch	CMP	8	ED	49.00	L	D-82	18	Inch	CMP	12	ED,CLC	42.60	L								
D-64	2@4	Feet	CMP's	1	ED	48.80	L	D-83	24	Inch	CMP	12	ED,CLC	42.50	L								
	Conc Grouted Riprap around Culvert							D-84	8	Feet	CMP/G	4	EDC	41.80	L								
D-65	2	Feet	CMP/G	2	SPC	48.65	L																
D-66	18	Inch	CMP	8	SP	48.45	L																
D-67	2	Feet	CMP	8	ED	47.80	L																
D-68	2	Feet	CMP	8	ED	47.60	L																
D-69	3	Feet	CMP/G	3	EDC	47.50	L																
D-70	24	Inch	CMP	10	ED	47.30	L																
D-71	24	Inch	CMP	1	ED	47.20	L																
D-72	24	Inch	CMP/G	1	EDC	47.10	L																
D-73	30	Inch	CMP	2	ED	47.05	L																
D-74	18	Inch	CMP	2	ED	46.80	L																
D-75	2@5	Feet	CMP's	3	EDC	45.20	L																

Treatment Codes: SP = Slope Protection; ED =Energy Dissipater; SPC = Slope Protection with concrete treatment; EDC = Energy Dissipater with concrete treatment; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CMPA = Corrugated Metal Pipe Arch; CMP/G = Corrugated Metal Pipe with grouted rock apron

Table G4. Little Goose Drainage Structures Treatment

Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)	Feature No.	Dia	Unit Meas	Mat'l Type	Drop to Res (Ft)	Treat-ment Code	River-mile	Bank (L/R)
D-1	24	Inch	CMP	3	ED	81.70	R	D-30	3	Feet	CMP	3	ED	71.60	R	D-59	24	Inch	CMP	20	ED,CLC	98.80	R
D-2	24	Inch	CMP	12	ED,CLC	81.50	R	D-31	24	Inch	CMP	5	SP	71.50	R	D-60	4	Feet	CMP	5	ED	98.50	R
D-3	30	Inch	CMP	5	ED	81.00	R	D-32	2@36	Inch	CMP	2	SP	71.20	R	D-61	24	Inch	CMP	4	ED	98.20	R
D-4	30	Inch	CMP	4	ED	80.95	R	D-33	36	Inch	CMP	3	SP	71.00	R	D-62	18	Inch	CMP	10	ED	97.90	R
D-5	2@4	Feet	CMP	4	ED	80.90	R	D-34	24	Inch	CMP	3	SP	70.90	R	D-63	4	Feet	CMP	5	ED	97.55	R
D-6	30	Inch	CMP	4	ED	80.50	R	D-35	36	Inch	CMP	3	SP	70.80	R	D-64	24	Inch	CMP	3	ED	97.50	R
D-7	3@30	Inch	CMP	2	ED	80.40	R	D-36	24	Inch	CMP	3	SP	83.10	L	D-65	24	Inch	CMP	4	ED	94.10	R
D-8	30	Inch	CMP	5	ED	79.90	R	D-37	18	Inch	CMP	6	SP	83.10	L	D-66	24	Inch	CMP	4	ED	93.50	R
D-9	4	Feet	CMP	0.5	ED	79.85	R	D-38	24	Inch	CMP	6	SP	83.10	L	D-67	24	Inch	CMP	6	ED	93.30	R
D-10	4	Feet	CMP	0.5	ED	79.80	R	D-39	24	Inch	CMP	15	ED,CLC	105.30	L	D-68	18	Inch	CMP	12	ED,CLC	92.80	R
D-11	18	Inch	CMP	2	ED	79.70	R	D-40	24	Inch	CMP	4	ED	105.40	L	D-69	24	Inch	CMP	8	ED	92.20	R
D-12	18	Inch	CMP	2	ED	79.40	R	D-41	18	Inch	CMP	4	ED	106.40	L	D-70	30	Inch	CMP	15	ED,CLC	91.40	R
D-13	18	Inch	CMP	2	ED	79.30	R	D-42	24	Inch	CMP	8	ED	107.10	L	D-71	24	Inch	CMP	4	ED	91.10	R
D-14	18	Inch	CMP	2.5	ED	79.00	R	D-43	18	Inch	CMP	8	ED	107.20	L	D-72	36	Inch	CMP	4	ED	90.05	R
D-15	18	Inch	CMP	3	ED	78.80	R	D-44	24	Inch	CMP	5	ED	107.10	R	D-73	24	Inch	CMP	5	ED	89.90	R
D-16	18	Inch	CMP	3	ED	78.50	R	D-45	24	Inch	CMP	5	ED	107.00	R	D-74	4	Feet	CMP	3	ED	89.75	R
D-17	24	Inch	CMP	3	ED	78.20	R	D-46	30	Inch	CMP	8	ED	105.10	R	D-75	18	Inch	CMP	3	ED	89.30	R
D-18	36	Inch	CMP	5	ED	75.50	R	D-47	30	Inch	CMP	8	ED	105.00	R	D-76	18	Inch	CMP	4	ED	88.90	R
D-19	3	Feet	CMP	3	ED	74.90	R	D-48	30	Inch	CMP	8	ED	104.80	R	D-77	4	Feet	CMP	4	ED	88.80	R
D-20	24	Inch	CMP	5	ED	74.70	R	D-49	30	Inch	CMP	6	ED	104.40	R	D-78	24	Inch	CMP	10	ED	88.45	R
D-21	18	Inch	CMP	2	ED	74.60	R	D-50	6	Feet	CMP	5	ED	103.50	R	D-79	4	Feet	CMP	5	ED	88.40	R
D-22	24	Inch	CMP	2.5	ED	74.50	R	D-51	36	Inch	CMP	6	ED	102.90	R	D-80	24	Inch	CMP	6	ED	88.00	R
D-23	36	Inch	CMP	3	ED	74.10	R	D-52	18	Inch	CMP	30	ED,CLC	102.80	R	D-81	30	Inch	CMP	4	ED	87.80	R
D-24	24	Inch	CMP	2	ED	73.95	R	D-53	24	Inch	CMP	25	SP,CLC	102.50	R	D-82	30	Inch	CMP	6	ED	87.65	R
D-25	24	Inch	CMP	2	ED	73.30	R	D-54	24	Inch	CMP	15	SP,CLC	102.40	R	D-83	2@3	Feet	CMP	4	ED	87.05	R
D-26	24	Inch	CMP	3	ED	72.90	R	D-55	3	Feet	CMP	25	ED,CLC	101.60	R	D-84	36	Inch	CMP	3	ED	86.90	R
D-27	3@3	Feet	CMP	3	ED	72.65	R	D-56	24	Inch	CMP	8	ED	99.90	R	D-85	18	Inch	CMP	2	ED	86.50	R
D-28	30	Inch	CMP	5	ED	71.80	R	D-57	24	Inch	CMP	2	ED	99.40	R	D-86	4	Feet	CMP	6	ED	84.20	R
D-29	3	Feet	CMP	3	ED	71.70	R	D-58	3	Inch	CMP	6	ED	99.00	R								

Treatment Codes: ED =Energy Dissipater; SP = Slope Protection; CLC = Clean Lower Culvert
Material Types: PVC = Polyvinylchloride; CMP = Corrugated Metal Pipe; Steel = Steel Pipe; ABS = Black Plastic Pipe; WS = Welded Steel; CHA = Concrete Half Arch; CMPA = Corrugated Metal Pipe Arch

G.3 Methodology for Drainage Structure Modifications

The study team evaluated existing culverts with respect to three conditions: 1) their position relative to the existing full reservoir water surface, 2) natural soil and rock slopes between drainage structure inverts and natural channel (full drawdown) elevations as taken from the U.S. Geological Survey quadrangle maps (scale 1:24,000), and 3) position of the culverts with respect to existing and potential drainage areas. Based on these evaluations, the team determined that four types of modification would be required to allow culverts and other drainage structures to continue to function after the reservoirs are drawn down.

Drain modifications consist primarily of extending a narrow riprap blanket along the drainage path below the outlet of the drainage structure to provide erosion protection of the underlying material. Additional actions that may be required for individual culverts include cleaning currently visible culverts and culverts submerged below normal reservoir level; diverting and combining drainage flows from two or more areas into a single culvert; or installing additional culverts. These modifications to the drainage structures are discussed in more detail below.

G.3.1 Extending a Riprap Blanket Below the Outlet of the Drainage Structure

The simplest and most common alternative for modifying the drainage structure is to extend a riprap blanket from the existing outlet of the drainage structure down to the restored natural river elevation. Two alternatives were selected for accomplishing this protection option. The primary difference in the two approaches is the steepness of slope from the drain discharge down to the restored natural river elevation. For both alternatives, the study team assumed that many of the areas of slope protection were accessible by existing roads or access roads developed for placement of riprap for railroad and highway embankments.

The first alternative (see Figure G1) would be used in areas where the natural slope abutting the embankment fill was steep and had a continuous elevation drop to the natural riverbed. These categories of drainage structures are given the treatment code SP (for "slope protection") in Tables G-1 through G-5. This approach applies a riprap blanket for slope protection for drainage structures extending from the outlet of the drainage structure to the natural river level. The width of the riprap blanket would vary depending on the diameter of the existing culvert, as follows: 1) culverts less than or equal to 900 mm (3 feet) would have a riprap blanket 3.0 m (10 feet) wide; 2) culverts sized from 900 mm to 1,500 mm (3 feet to 5 feet) would have a riprap blanket 4.6 m (15 feet) wide; and 3) culverts greater than 1,500 mm (5 feet) would have a riprap blanket 6.1 m (20 feet) wide.

The thickness of the riprap blanket and outlet pad would be 0.6 m (2 feet) for riprap placed by conventional placement methods. Riprap would be well graded from a maximum size of 1.5 times the average rock size, or 0.3 m (1 feet), to 25.4-mm (1-inch) spalls suitable to fill the voids between the rocks.

In developing this modification plan, the study team assumed the maximum vertical drawdown for the four projects would be 27 m (90 feet) near the dams. Areas that would require longer lengths of slope protection are typically located either nearer the dams where the greatest reservoir drawdown would occur, or high up on the slope. Most of the rock in this slope protection category would be larger than the material of the natural or embankment slope on which it would be placed.

Some areas requiring slope protection, specifically those next to concrete headwalls and other large diameter drainage structures, have a matrix of cement grout or gunite binding the rock together, thus creating a more erosion-resistant surface. The energy dissipative effects of the ungrouted rocks may not be sufficient to protect the slope downstream of the culvert exits on these structures. Consequently,

grouting of the riprap slope protection would be required in these locations. These drainage structures are given the treatment code SPC (for "slope protection with concrete treatment") in Tables G-1 through G-5.

The second treatment alternative (see Figure G2) is used in areas where the natural elevation contours from the culvert to the original riverbed are fairly flat, having either minimal elevation loss or elevation loss spread over a long distance. In this situation, a riprap blanket with the same rock gradation as in the first treatment alternative would be placed for a distance of 6.1 m (20 feet) along the drainage path, beginning at the outlet of the culvert, to dissipate energy. The thickness of the energy dissipater would be 0.6 m (2 feet). The width of the riprap blanket would vary from 3 m (10 feet) for drains with a maximum diameter of 0.9 m (3 feet), up to a maximum of 6.1 m (20 feet) for the larger diameter structures. These structures are assigned the treatment code ED (for "energy dissipater") in Tables G-1 through G-5. If concrete treatment is required, the designated treatment code assigned is EDC (for "energy dissipater with concrete treatment").

In the tables, the treatment code NT (for "no treatment") has been used for some of the culverts. At some locations, a lower culvert is positioned at the same location or within 0.16 kilometers (0.1 mile) of a higher culvert (greater than 3.0 m [10 feet] above reservoir level). To minimize slope protection, the study team assumed that, in these situations, the lower culvert would pass flows and the upper culvert would probably remain "high and dry." Therefore, no treatment was planned for the higher culverts. Existing documents indicate that some of the lower culverts were sized for temporary construction and, therefore, might not have adequate size and flow capacity. However, additional lower culverts were noted on the as-constructed drawings. Consequently, the study team determined that the number of upper culverts requiring slope protection is likely to be conservative.

G.3.2 Cleaning Culverts

Existing culverts that are visible above the present normal reservoir water surface might be plugged. Cleaning would be needed to allow these culverts to properly function. Based on its field observations, the study team estimated 25 percent of existing culverts for all the reservoirs would require cleaning.

The team's review of as-constructed contract documents for railroad and highway relocations at both the Little Goose and Ice Harbor dams identified currently submerged culverts that also would require cleaning. At Little Goose, the number of submerged culverts was found to total 40 percent of the number of culverts visible above the reservoir water surface. Because of the similarity in railroad relocations and construction at both Little Goose and Lower Granite reservoirs, the team also assumed that the number of submerged culverts at Lower Granite would be 40 percent of the visible culverts. At Ice Harbor Reservoir, the number of submerged culverts was found to be 7 percent of the total number of visible culverts. This assumption was also used at Lower Monumental Reservoir because of the similarity of Lower Monumental railroad relocation and construction with that at Ice Harbor. Review of design memoranda for Ice Harbor Dam indicates that some of the lower culverts were intentionally plugged after construction. The study team conservatively assumed all of these lower elevation culverts would require cleaning (see Figure G3).

Based on these assumptions, 28 culverts would require cleaning at Ice Harbor Reservoir, 28 culverts at Lower Monumental Reservoir, 60 culverts at Little Goose Reservoir, and 102 culverts at Lower Granite Reservoir, resulting in a total of 218 culverts needing to be cleaned. The estimated average length of culvert to be cleaned is 30.5 m (100 feet).

G.3.3 Adding Culverts

Drawdown conditions may create new or re-established drainage paths that require new culverts to be installed at lower elevations to allow for proper drainage. The study team estimated that a quantity of

new culverts equal to 10 percent of the existing culverts would need to be installed at each of the reservoir sites. These additional culverts would be constructed by the cut and cover method.

G.3.4 Diverting and Combining Drainage Flows

In some of the locations that require installation of a new drain, it may be more cost-effective to collect water flows from adjacent run-off areas and route the discharge to the Snake River via one new outfall (Figure G4). The receiving discharge structure might need to be enlarged to accommodate combined flows. Pipe jacking or horizontal boring through the existing embankments would install these new enlarged drains, assuming that an open cut in the existing embankment is not practical. The casing of the horizontal boring would be used as the drain conduit. No additional pipe would be needed inside the casing.

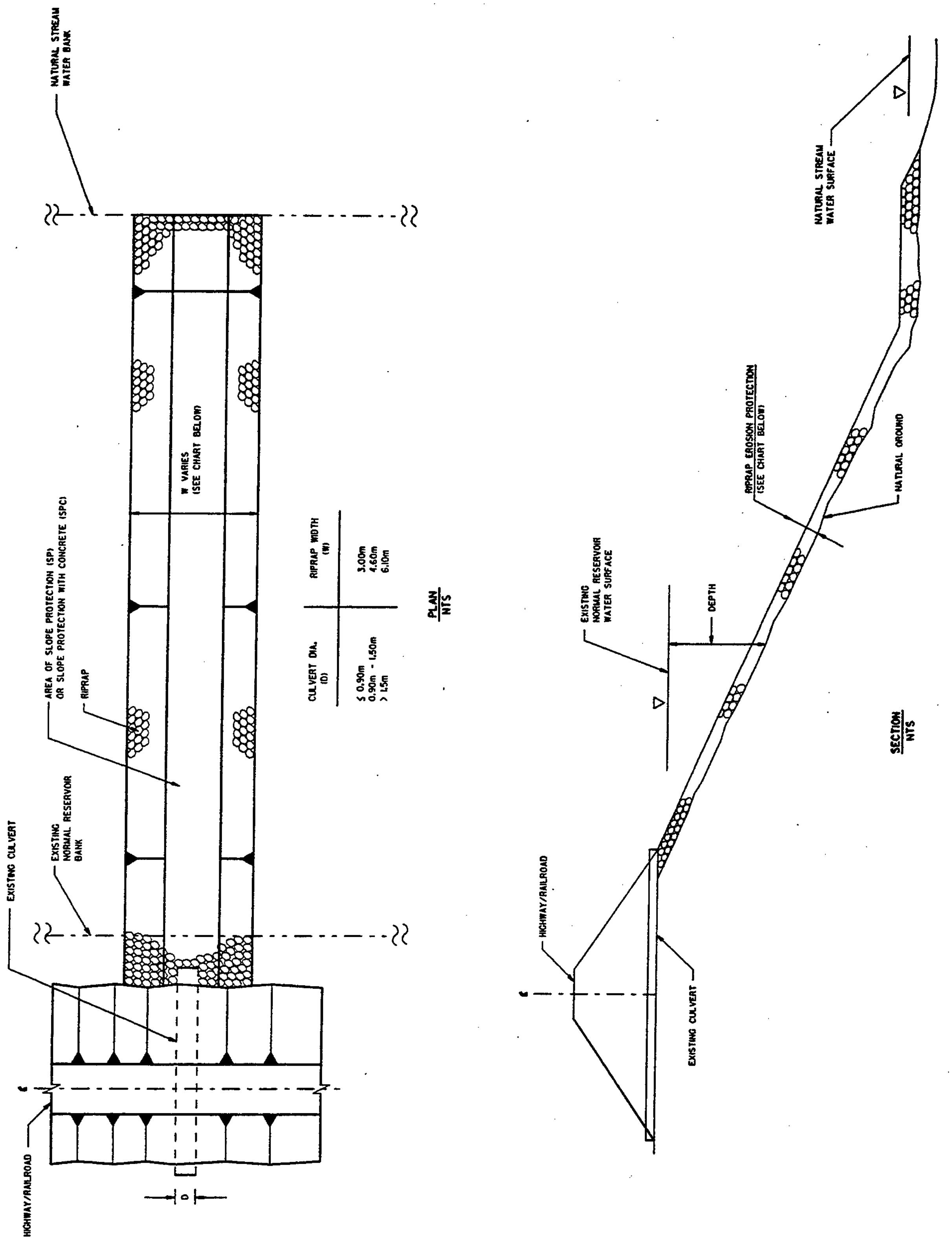
The study team could not determine the number of sites where this alternative would be more cost effective than cleaning or replacement of individual culverts, but estimated a quantity of 3 percent of the total number of culverts identified for cost estimating purposes.

G.4 Construction Scenario

Modifications to the existing drainage structures would provide some logistical challenges. Because the drains are spaced far apart, have difficult land access, and require placement of narrow strips of riprap extending down steep slopes. Overland access is possible to all drainage structures via haul roads developed for transportation and placement of riprap. Additional rock would be hauled for drainage structure modifications and placement done concurrently or immediately following riprap placement in a specific reach.

Placement would be by conventional methods. An excavator would prepare the slope, if necessary, for riprap placement. Some excavation may be necessary to form a channel to limit the dispersion of drainage flows and thereby minimize damage to the embankment slope. Subsequent placement of concrete to stabilize riprap would be done with a concrete pump and appropriate nozzle if pneumatic application is necessary.

The source of acceptable riprap materials is discussed in Annex F of this report and applies to any material placed for drainage protection.



US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

DRAINAGE STRUCTURE SLOPE PROTECTION
PLAN AND SECTION

Figure: G1

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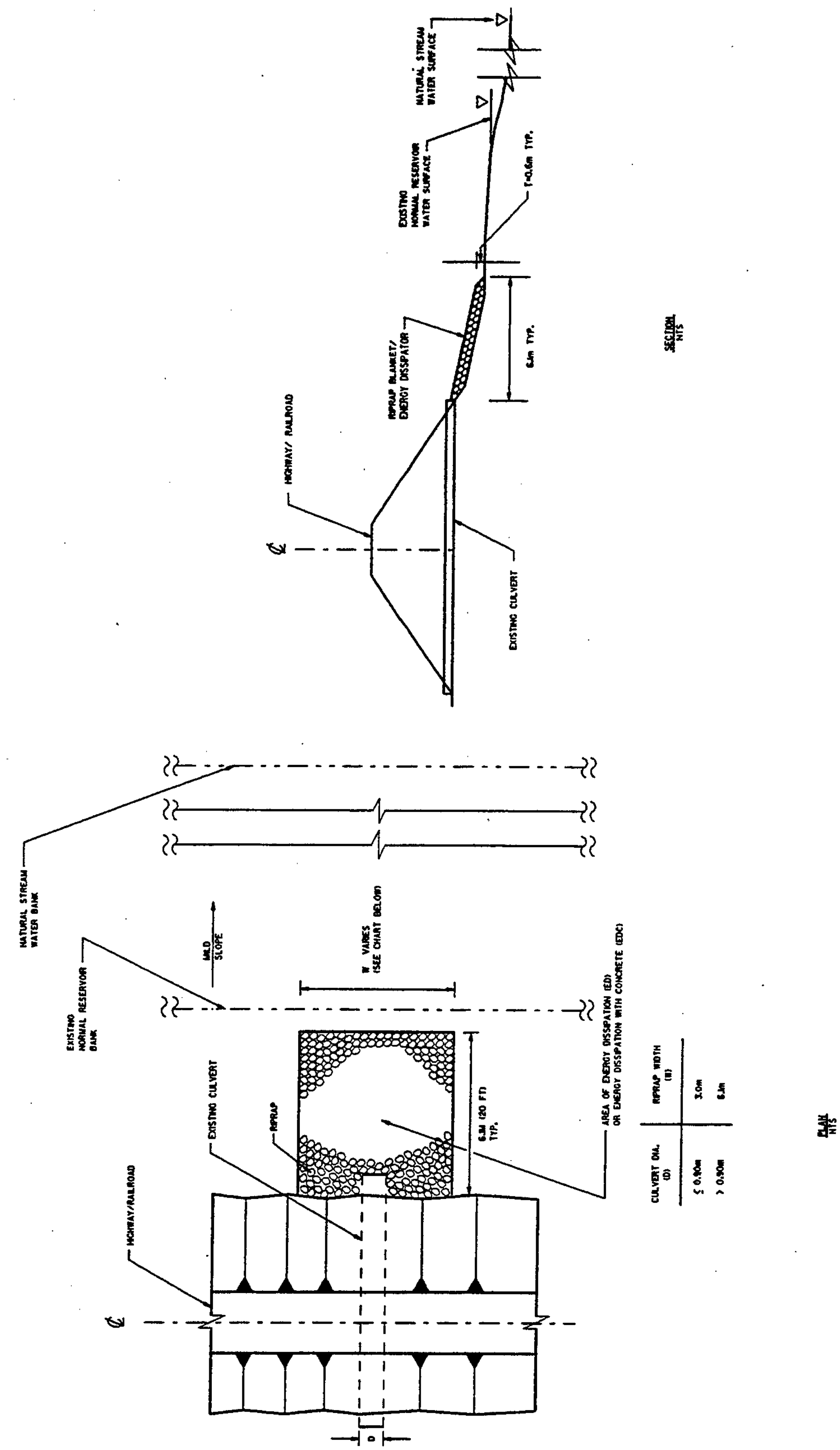

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

DRAINAGE STRUCTURE ENERGY DISSIPATOR
PLAN AND SECTION

Figure: G2

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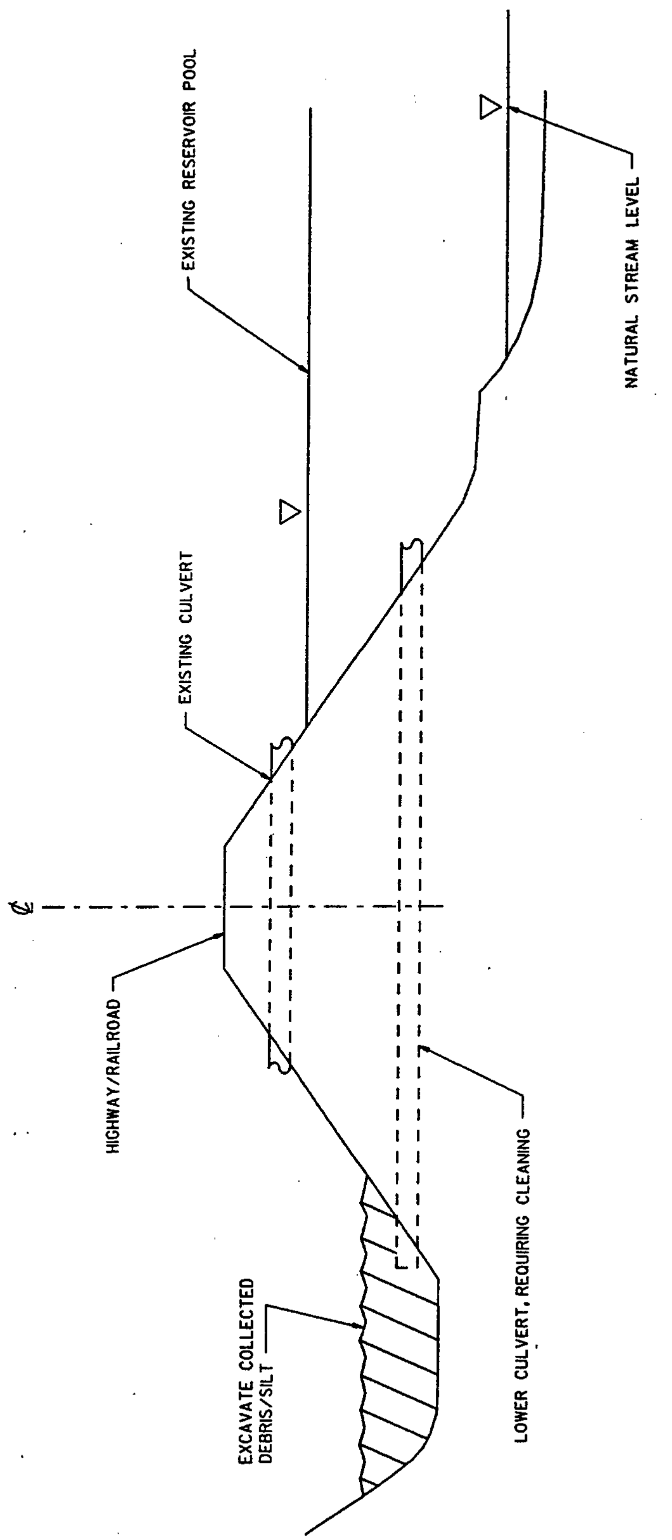

CLEAN LOWER CULVERT - CROSS SECTION

NOT TO SCALE

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LOWER CULVERT CLEANING
CROSS SECTION

Figure:
G3

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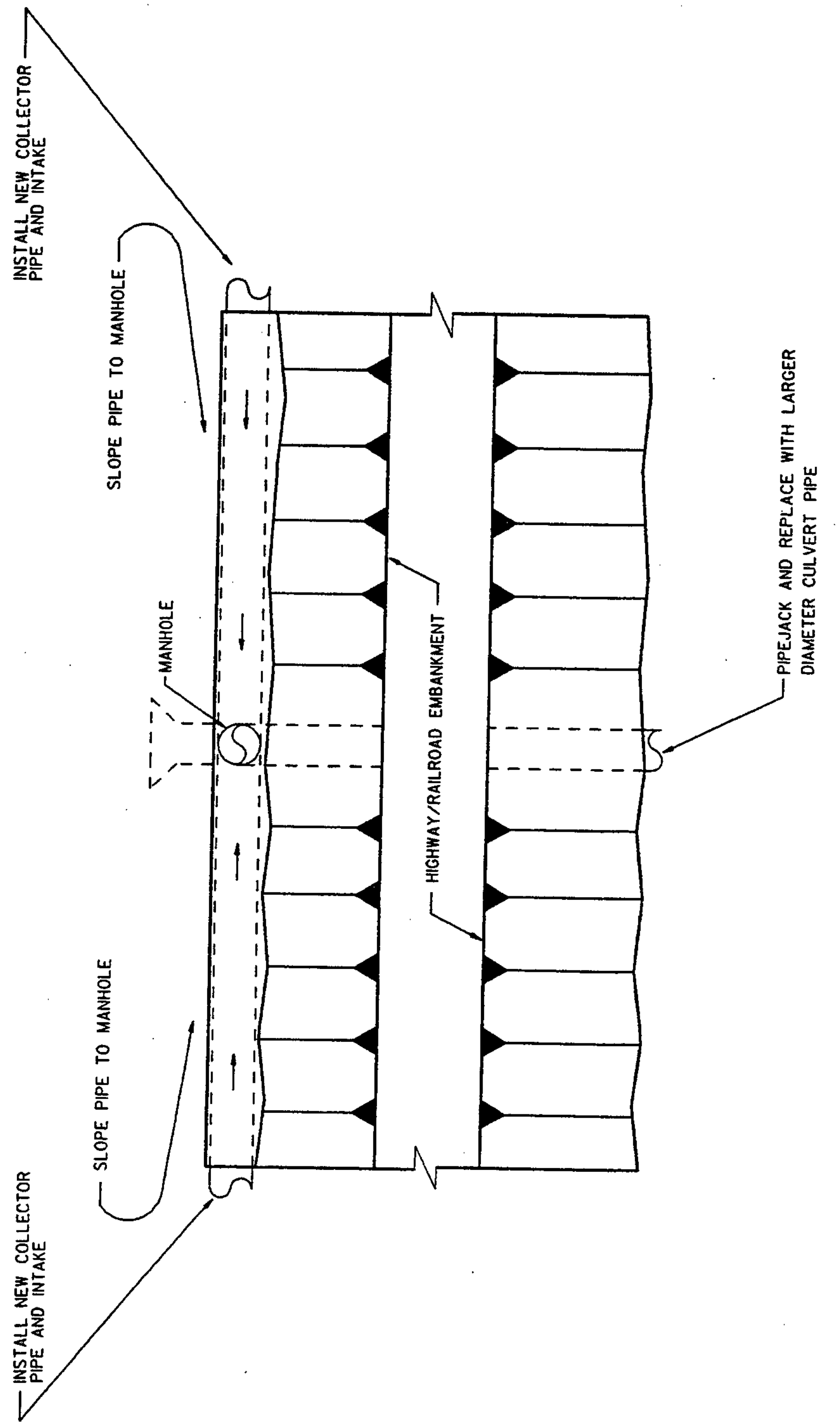

DIVERT/COMBINE DRAINAGES - PLAN VIEW

NOT TO SCALE

US Army Corps
of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

DIVERT/COMBINE DRAINAGES
PLAN VIEW

Figure:
G4

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Annex H

Railroad and Roadway Damage Repair Plan

Annex H: Railroad and Roadway Damage Repair Plan

H.1 Introduction

This portion of the study addresses of the potential effects of drawdown on railroad and roadway embankments from the Snake River's confluence with the Columbia River to the Idaho state line. Those effects are settlement and slope stability directly impacted by the drawdown of the reservoir. Problems and anticipated modifications required to resist the erosive forces of the river on the embankments are described in Annex F.

There is no doubt that many of the railroad and highway embankments will be damaged as a result of rapid reservoir drawdown. As drawdown occurs, areas of the embankments along the river are anticipated to fail due to steep slopes, saturated soils, and pore pressure increase. This annex describes the critical elements that contribute to embankment failures from rapid drawdown. It summarizes the observations from the 1992 test Drawdown and, from those observations, projects damages resulting from a full reservoir drawdown. It discusses the necessity and impacts of the selected drawdown rate.

H.2 Review of 1992 Drawdown

A test of the reservoir drawdown concept was performed in March 1992, using Lower Granite and Little Goose dams. The purpose of the test was to gather information regarding the effects of substantially lowering existing reservoirs. The drawdown test was scheduled to be completed within the month of March in order to minimize potential negative impacts to Snake River migrating fish. On March 1 the Lower Granite reservoir was drafted from its starting point of normal minimum operating pool (elevation 223.4 meters (733 ft)) at a rate of 0.6 meters per day for 14 days. Elevation 214.9 meters (705 ft) was achieved on March 15. During subsequent phases. Little Goose reservoir was lowered a total of 3.8 meters and Lower Granite Reservoir was further lowered to elevation 212.4 meters (697 feet) for a total drawdown of 11.0 meters.

During the drawdown the Corps monitored road and railroad embankments along the two reservoirs for potential problems. The following damage on the Lower Granite reservoir was reported:

- Camas Prairie Railroad (CPRR) embankment experienced cracking, movement and track misalignment;
- Whitman County Road 9000 embankment experienced extensive movement and cracking in 33 areas (cracks varied in width from a few millimeters to 0.4 meters, and some over 60 meters in length) and damage to roadway and guardrail;
- State Highway 193 between Steptoe Canyon and Red Wolf bridge experienced cracking and movement;
- U.S. Highway 12 had two small slides (generally minor) near Red Wolf Marina and soil piping was noted; and
- Cracking and movement of the road and railroad embankments disturbed many survey monuments.

It was noted that most of the sliding activity associated with the drawdown occurred within slopes consisting of natural deposits of silts, sands, and gravels. For the purposes of this study, stability of natural slopes was not addressed, and efforts focused on man-made embankments. Drawdown of each reservoir of up to 30 meters cannot be assumed to occur without embankment failures.

H.3 Embankment Geometry and Material Considerations

The key to understanding how embankments will behave under drawdown conditions is to understand the embankment materials. Embankments constructed from materials that are so "free-draining" that the soil saturation level falls quickly will have increased stability under drawdown conditions. Stability is decreased if the soil saturation level lags behind the reservoir drawdown level. Therefore, the rate of drawdown associated with a minimal lag is related to the "free draining" ability of embankment materials. Greater permeability and porosity of soils results in a greater ability of the material to be "free draining." Although a material may be free draining, the rate of reservoir drawdown may be too fast, resulting in a greater saturation level lag and reduced embankment stability.

The man-made embankments along the lower Snake River are, in general, constructed from locally borrowed materials, and were not subject to the same quality control efforts (grain size and compaction control) which were used in construction of major embankment dams. Also, internal drainage features such as pipes or clean stone drains were not incorporated into the designs. According to railroad and roadway relocation reports and drawings, many embankments were constructed from "random fill" or "granular fill" materials. Compaction was probably used in placing these materials, but it is not clear how much compactive effort was used and what methods were employed. The nature of "random fill" available for borrow in the vicinity of the lower Snake River varies, although the material is predominately sand and gravel with varying amounts of fines (silts and clays passing the No. 200 sieve) and cobbles. The CPRR relocation report (Lower Granite DM 9.2) states that embankment foundations along the relocated alignments consists of bedrock or materials described as relatively clean talus rock, silty talus rock, alluvial material, and wind-deposited sand and silts. Similar materials were used for construction of the relocated road and railroad embankments.

The amount of fines controls the ability of an embankment material to be "free draining," and the amounts of fines in silty talus rock and wind-deposited sands and silts could be significant enough to preclude free draining conditions. Alluvial materials obtained from local terrace gravel deposits and clean talus rock materials likely consist of a predominantly granular mixture of sand, gravel, and cobbles, with a lower percentage of fines than the silty materials. Although aeolian silt often exists on the ground surface of the terrace gravel deposits, it is not likely that significant amounts of fines are present in the alluvial random fill mixtures. The ability of the embankments to be free draining, and therefore more stable during drawdown, depends on the borrow source used to construct the embankments.

Man-made embankments were generally constructed with slopes of 2H:1V, with riprap or rockfill slope protection within the normal reservoir surface operating range. Some embankments, particularly on the Ice Harbor reservoir, have buttress fills against the toe of embankments with slopes of 2.5 H:1V to 3H:1V. The embankments along the reservoirs have various top and toe elevations, and the drawdown range will vary from approximately 30 meters just upstream of each dam to nearly no drawdown, or possibly a slight increase in water level, just downstream of each dam. There are many embankment and drawdown rate configurations, and when the variations in embankment geometry, material types and compaction criteria are considered, there are an infinite number of material parameter and geometric combinations.

H.4 Rate of Reservoir Drawdown

The man-made embankments along the four lower Snake River reservoirs were constructed by various entities (including the federal government, state transportation department, and railroad companies) over an extended period of time. Embankment characteristics which vary include the method of embankment construction, embankment geometry, materials used in the embankments, surrounding land topography, embankment foundation materials, and vertical distance of drawdown from the normal reservoir surface elevation. All of these characteristics result in embankments which will behave differently under a

drawdown scenario. Behavior may vary from no visible movement or damage to few tension cracks and minor movement or sloughing, to the extreme case of slope failure with extensive movement.

The rate of reservoir drawdown is an important parameter in establishing the schedule for overall embankment dam removal and reservoir drawdown. There are several biological and weather factors which influence the beginning, end, and duration of drawdown. The primary constraint in determining the rate of drawdown is the time period during which the reservoir must be lowered and the embankment removed. Reservoir evacuation cannot begin in any year prior to 1 August. This is because the spring runoff flows extend into June and July and downstream fish migration continues until this time. By January of any year the probability of high flows in the river increases dramatically. These beginning and end point constraints require that the drawdown to be done during this 5-month period. This time is further reduced to allow sufficient time to excavate the embankment and remove cofferdams.

The drawdown rate will be controlled at each dam by the spillway and powerhouse gates. Consequently, a nominal drawdown rate of 0.6 meters (2 ft) per day has been assumed for feasibility level construction planning. While some latitude may be possible as designs and schedules are further developed, the drawdown rate of 0.6 meters/day may only be slightly reduced.

H.5 Methods

The location and extent of embankment failures is extremely difficult to predict based on the uncertainty and variability of materials used in constructing the embankments. However, embankment damage data from the 1992 drawdown of Lower Granite was useful in making such predictions. Table H1 summarizes the specific areas where damage was observed after the 1992 test drawdown. A rational methodology was desired to determine potential damages and subsequent repairs. To estimate the potential for road and railroad embankment failures from observed embankment distress, the study team made the following assumptions:

1) Drawdown would remove hydrostatic support from saturated materials.

2) The sections anticipated to undergo settlement are those that are in similar physical positions (height and distance) as the sections that exhibited settlement along the Lower Granite Reservoir during the 1992 drawdown.

3) The anticipated failure type and characteristics are theoretical and are based on an infinite-slope analysis. Some parameters are based on field observation, and some are based on information resources such as topographic maps and aerial photographs.

The team developed materials estimates for making repairs to the road and railroad embankments using the following assumptions:

1) The dimensions for road and railroad cross sections were assumed to be the same as the typical sections used for the road and railroad relocations prior to reservoir establishment. The team also assumed that road and railroad embankments would be constructed with materials meeting current standards.

2) Material sources were selected from existing sources identified on maps and aerial photographs. All sources were assumed to be available for use and no ownership issues were considered. Haul distances were based on sources shown on maps and aerial photographs.

Table H1. Measurements of Distress from Observations of 1992 Drawdown

Station	Station Location	Feature	Description	Natural Slope	Distance From River (ft)	Height Above River (ft)	Embankment Slope	Materials
2431+14	Rd. 9000	Pavement Crack	149 ft long, 1 inch wide	30%	50	30	60%	0 ft to 14 ft: silt with scattered rock fragments
2452+26	Rd. 9000	Pavement Crack	58 ft long, 1/2 inch wide	17%	50	20	40%	0 ft to 15 ft: rock fragments in sandy silt matrix
2457+54	Rd. 9000	Pavement Crack	19 ft long, 1/4 inch wide	18%	50	20	40%	0 ft to 15 ft: fine sandy silt with rock fragments
2552+58	Rd. 9000	Pavement Crack	422 ft long, 10 inch wide	4%	50	20	40%	0 ft to 15 ft: fine sandy silt with rock fragments
2605+38	Rd. 9000	Pavement Crack	248 ft long, 1 ft wide	60%	30	20	60%	0 ft to 15 ft: interbedded silt and sand
2605+38	Rd. 9000	Pavement Crack	63 ft long, 1/4 inch wide	60%	30	20	60%	0 ft to 15 ft: interbedded silt and sand
2626+50	Rd. 9000	Pavement Crack	341 ft long, 9 inch wide	6%	50	20	40%	0 ft to 14 ft: silt with scattered rock fragments
2637+06	Rd. 9000	Pavement Crack	154 ft long, 3 inch wide	13%	20	10	50%	0 ft to 15 ft: silt with scattered rock fragments
2684+58	Rd. 9000	Pavement Crack	80 ft long, 1/4 inch wide	8%	50	20	40%	0 ft to 14 ft: sandy silt
2710+98	Rd. 9000	Pavement Crack	24 ft long, 6 inch wide	27%	50	20	40%	0 ft to 15 ft: silt with scattered rock fragments
2742+66	Rd. 9000	Pavement Crack	221 ft long, 3/4 inch wide	50%	30	20	65%	0 ft to 14 ft: silt with scattered rock fragments
2753+22	Rd. 9000	Pavement Crack	45 ft long, 2 inch wide	17%	30	20	65%	0 ft to 14 ft: rock fragments in silty and ash matrix
2753+22	CPRR	Pavement Crack	197 ft long, 15 inch wide	17%	30	20	65%	0 ft to 14 ft: rock fragments in silty and ash matrix
2758+50	CPRR	Pavement Crack	33 ft long, 6 inch wide	30%	30	20	65%	0 ft to 14 ft: rock fragments in silty and ash matrix
2758+50	CPRR	Pavement Crack	51 ft long, 7 inch wide	30%	30	20	65%	0 ft to 14 ft: rock fragments in silty and ash matrix
2763+78	Rd. 9000/ CPRR	Pavement Crack	191 ft long, 6 inch wide	25%	40	20	50%	0 ft to 40 ft: interbedded silt and sand
2763+78	Rd. 9000	Pavement Crack	48 ft long, 2 inch wide	25%	40	20	50%	0 ft to 40 ft: interbedded silt and sand
2779+62	Rd. 9000	Pavement Crack	81 ft long, 6 inch wide	18%	50	20	40%	0 ft to 3 ft: sand and gravel, 3 ft +: bedrock
2784+90	Rd. 9000	Pavement Crack	118 ft long, 13 inch wide	12%	40	20	50%	0 ft to 14 ft: rock fragments in silty matrix

Table H-1, continued. Measurements of Distress from Observations of 1992 Drawdown

Station	Station Location	Feature	Description	Natural Slope	Distance From River (ft)	Height Above River (ft)	Embankment Slope	Materials
2784+90	Rd. 9000	Pavement Crack	102 ft long, 4 inch wide	12%	40	20	50%	0 ft to 14 ft: rock fragments in silty matrix
2784+90	Rd. 9000	Pavement Crack	228 ft long, 13 inch wide	12%	40	20	50%	0 ft to 14 ft: rock fragments in silty matrix
2790+18	Rd. 9000	Pavement Crack	289 ft long, 7 inch wide	40%	50	20	40%	0 ft to 14 ft: rock fragments in silty matrix
2800+74	Rd. 9000	Pavement Crack	313 ft long, 11 inch wide	17%	50	20	40%	0 ft to 14 ft: rock fragments in silty matrix
2806+02	Rd. 9000	Pavement Crack	116 ft long, 9 inch wide	40%	30	20	65%	0 ft to 14 ft: rock fragments in silty matrix
2806+02	Rd. 9000	Pavement Crack	254 ft long, 10 inch wide	40%	30	20	65%	0 ft to 14 ft: rock fragments in silty matrix
2811+30	Rd. 9000	Pavement Crack	241 ft long, 1 inch wide	10%	50	20	40%	0 ft to 14 ft: rock fragments in silty matrix
2816+58	Rd. 9000	Pavement Crack	56 inch long, 1/8 inch wide	20%	60	30	50%	0 ft to 14 ft: rock fragments in silty matrix
2849+94	Rd. 9000	Pavement Crack	50 ft long, 1/4 inch wide	30%	50	20	40%	0 ft to 14 ft: rock fragments in silty matrix
2890+50	Rd. 9000	Pavement Crack	204 ft long, 1/4 inch wide	26%	30	10	30%	0 ft to 14 ft: rock fragments
2901+06	Rd. 9000	Pavement Crack	253 ft long, 5 inch wide	19%	40	15	40%	0 ft to 14 ft: rock fragments in silty matrix
2948+58	Rd. 9000	Pavement Crack	15 ft long, 1/4 inch wide	15%	40	15	40%	3 ft to 6 ft: gravel 6 ft to 12 ft: silt
2953+86	CPRR	Pavement Crack	123 ft long, 6 inch wide	4%	150	20	13%	volcanic ash, silt, and sand
2959+14	CPRR	Pavement Crack	30 ft long, 4 inch wide	7%	50	20	40%	0 ft to 4 ft: talus and colluvium 4 ft+: bedrock
2959+14	Rd. 9000	Pavement Crack	162 ft long, 14 inch wide	7%	50	20	40%	0 ft to 4 ft: talus and colluvium 4 ft+: bedrock
2959+14	Rd. 9000	Pavement Crack	758 ft long, 14 inch wide	5%	50	20	40%	0 ft to 35 ft: interbedded silt and sand
2964+42	Rd. 9000	Pavement Crack	278 ft long, 2 inch wide	18%	60	20	30%	0 ft to 35 ft: interbedded silt and sand

3) The embankment repair quantities were assumed to be cumulative for each project.

4) Since the water level would be far below the structures, the team assumed that riprap would only be needed for shoreline protection in the active water surface zone.

5) Quantities were based on the following thicknesses:
   - Asphalt surfacing - 75 millimeters (mm)
   - Surface course - 150 mm
   - Base course - 300 mm
   - Ballast - 900 mm
   - Sub-ballast - 300 mm.

Combinations of theoretical and practical methods were used to evaluate potential railroad and roadway damage during drawdown. Practical methods were based on observations made during the 1992 Lower Granite Reservoir drawdown. The drawdown test section consisted of Whitman Co. Road No. 9000 and the Camas Prairie Railroad along the Lower Granite Reservoir (Steptoe Canyon to Wawawai Canyon). It appeared that many failures occurred along the contact between the structure fill and the natural foundation material. At other locations, it was evident that the failure extended into the foundation material. Therefore, both modes of failure had to be taken into account. The measurements taken at the time of the observations are summarized in Table H1.

Also, from the observations along the test section, it was evident that nearly all failures occurred at locations that were within 15 meters (m) horizontal distance and 6 m vertical distance of the reservoir perimeter, and on slopes less than 50 percent (greater than 50 percent would indicate shallow bedrock and greater stability). Therefore, the study team concluded that sections along the river in similar positions with similar physical characteristics would display a similar response. The team also assumed that sections at a horizontal distance of 15 m to 30 m and vertical distance greater than 6 m from the reservoir would display only about 10 percent of the failures of the more closely adjacent sections. The areas of settlement within the test section along the Lower Granite Reservoir are marked on 1 inch = 1000 feet maps, contract drawing maps, and copies of aerial photographs in the *1992 Reservoir Drawdown Test, Lower Granite and Little Goose Dam* (Corps 1993). Using U.S. Geological Survey, 7.5 minute, 1:24,000 scale quadrangle maps, the study team delineated the sections in both modes of failure types and measured the approximate distance in feet of each.

The study team estimated that a total of 68 potential failure areas could result. These anticipated failure areas are shown in Table H2.

The study team also used a theoretical approach to determine the possibility of failure of natural slopes. Using the infinite slope equations for slope stability, the team calculated the factors of safety according to the following parameters:

- Slopes: 10 to 50 percent
- Soil: silt (classified as ML) with scattered cobbles and boulders
- Angle of internal friction: 30 degrees
- Height of phreatic surface above bedrock: 0.0 m to 4.5 m
- Saturated density: 1954 kilograms per cubic meter (kg/m^3)
- Moist density (10 percent moisture content): 1666 kg/m^3
- Depth to bedrock: 4.5 m.

Table H2. Potential Failure Areas Resulting from a Permanent Drawdown

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
Ice Harbor Reservoir								
BNRR	North Bank	S18, T9N, R32E	121.9	Low	1.4	1.0	107.8	4.6
BNRR	North Bank	S18, T9N, R32E	182.9	High	20.6	1.0	1617.1	3.8
BNRR	North Bank	S18, T9N, R32E	91.4	Low	1.0	1.0	81.0	3.0
BNRR	North Bank	S18, T9N, R32E	152.4	High	17.2	1.0	1349.5	2.4
BNRR	North Bank	S7, T9N, R32E	304.8	Low	3.4	1.0	269.9	2.3
BNRR	North Bank	S8, T9N, R32E	487.7	High	55.0	1.0	4320.0	1.2
BNRR	North Bank	S4,5, T9N, R32E	1066.8	Low	12.0	1.0	945.0	1.2
BNRR	North Bank	S4, T9N, R32E	182.9	High	20.6	1.0	1620.2	2.3
BNRR	North Bank	S3,T9N, R32E	335.3	Low	3.4	1.0	269.9	2.4
BNRR	North Bank	S34,T10N, R32E	152.4	Low	1.7	1.0	134.6	3.3
BNRR	North Bank	S26,T10N, R32E	152.4	High	17.2	2.0	1349.5	1.7
BNRR	North Bank	S26,T10N, R32E	1066.8	Low	12.0	2.0	945.0	2.4
BNRR	North Bank	S23,S26,T10N, R32E	1371.6	Low	15.5	2.0	1215.7	0.9
BNRR	North Bank	S24,T10N, R32E	274.3	Low	3.1	2.0	243.9	0.6
BNRR	North Bank	S13,T10N, R32E	274.3	High	30.9	3.0	2429.1	0.3
BNRR	North Bank	S12,T10N, R32E	792.5	Low	8.9	3.0	701.1	2.1
BNRR	North Bank	S4,T10N, R33E	701.0	Low	6.2	3.0	488.6	6.7
BNRR	North Bank	S27,34, T11N, R33E	1371.6	Low	15.5	3.0	1215.7	14.6
BNRR	North Bank	S14,23, T11N, R33E	670.6	Low	7.6	4.0	593.3	11.0
Burr Cyn. Rd.	North Bank	S19, T12N, R34E	121.9	Low	1.4	4.0	107.8	4.6
Burr Cyn. Rd.	North Bank	S18, T12N, R34E	426.7	High	24.1	4.0	1890.1	3.7
Burr Cyn. Rd.	North Bank	S 8,17, T12N, R34E	548.6	High	61.9	4.0	4858.3	2.4
Wilson Cyn. Rd.	North Bank	S4,9, T12N, R34E	2438.4	High	275.1	4.0	21596.1	1.2
Gravel Road	South Bank	S19, T9N, R32E	609.6	High	68.8	13.0	5398.8	0.6

Table H-2, continued. Potential Failure Areas Resulting from a Permanent Drawdown

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
UPRR	South Bank	S9, T9N, R32E	1828.8	Low	20.6	14.0	1620.2	0.9
UPRR	South Bank	S3,4, T9N., R32E	1828.8	High	206.3	14.0	16196.5	1.2
UPRR	South Bank	S2, T9N., R32E	1676.4	High	189.1	14.0	14847.0	3.7
UPRR	South Bank	S36, T10N, R32E	396.2	High	44.7	14.0	3508.0	4.9
UPRR	South Bank	S8, T10N, R33E	1524.0	Low	1.7	15.0	133.8	1.2
UPRR	South Bank	S34, T11N, R33E	701.0	Low	7.9	15.0	619.3	3.7
UPRR	South Bank	S26, T11N, R33E	762.0	Low	8.6	16.0	675.1	2.4
UPRR	South Bank	S24, T11N, R33E	609.6	High	68.8	16.0	5398.8	0.2
UPRR	South Bank	S12, T11N, R33E	3048.0	High	343.8	16.0	26995.0	2.3
UPRR	South Bank	S30,31, T12N, R33E	3048.0	Low	3.4	19.0	270.7	0.9
UPRR	South Bank	S17,19, T12N, R34E	5181.6	High	402.3	17.0	31590.2	2.1
UPRR	South Bank	S8,9, T12N, R.34E	1219.2	Low	13.7	17.0	1079.6	0.5
Lower Monumental Reservoir								
UPRR	South Bank	S35,36, T13N, R34E	1828.8	High	206.3	18.0	16201.9	0.8
UPRR	South Bank	S30,36, T13N, R34, 35E	1219.2	High	137.5	20.0	10793.1	0.9
UPRR	South Bank	S26,27,.28,29, T13N, R35E	7315.2	High	825.1	21, 22	64783.8	1.8
UPRR	South Bank	S21, T13N, R36E	304.8	High	34.4	23.0	2699.0	1.6
UPRR	North Bank	S2,3, T12N, R37E	1524.0	Low	17.2	5.0	1349.5	4.0
UPRR	North Bank	S36, T13N, R37E; S31, T13N, R38E	1524.0	Low	17.2	5.0	1349.5	1.2
HWY. 261	South Bank	S3,4, T12N, R37E	609.6	High	68.8	25.0	5398.8	4.7
Deadman Creek Rd.	South Bank	S32,33, T13N, R38E	609.6	High	68.8	26.0	5398.8	1.8
Little Goose Reservoir								
CPRR	North Bank	S22,23, T13N, R38E		Low	5.2	5a	406.8	2.7
CPRR	North Bank	S22,23, T13N, R38E	1066.8	High	120.4	5a	9452.7	2.7

Table H2, continued. Potential Failure Areas Resulting from a Permanent Drawdown

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
CPRR	North Bank	S24, T13N, R38E	243.8	Low	2.7	5b	215.6	2.4
CPRR	North Bank	S19,24, T13N, R38E	1066.8	High	120.4	5b	9453.5	0.9
CPRR	North Bank	S20,21, T13N, R38E	1524.0	High	171.9	5b	13497.5	2.4
CPRR	North Bank	S22, T13N, R38E	457.2	High	51.8	5b	59398.7	4.6
CPRR	North Bank	S7,11,14,23, T13N, R39,40E	2590.8	High	756.5	5b	59399.5	8.5
CPRR	North Bank	S7,12,14,23, T13N, R39,40E	1219.2	Low	1.0	5b	81.0	10.4
CPRR	North Bank	S13,14,22,23,27, T14N, R40E	4876.8	High	550.2	6.0	43198.4	3.1
CPRR	North Bank	S13,17,18, T14N, R40,41E	1524.0	High	171.9	7.0	13497.5	3.1
CPRR	North Bank	S15,16,17, T14N, R41E	3962.4	High	446.8	9.0	35083.7	1.8
CPRR	North Bank	S20, T14N, R42E	1219.2	High	137.5	9.0	10798.4	8.5
CPRR	North Bank	S20,21, T14N, R42E	914.4	Low	10.3	9.0	808.9	8.5
CPRR	North Bank	S13,14,23, T14N, R42E	3048.0	Low	343.8	9.0	26995.0	15.3
CPRR	North Bank	S13,18,19, T14N, R42E,43E	1828.8	High	206.3	10.0	16197.3	14.6
Hwy 127	South Bank	S9, T13N, R40E	1219.2	High	137.5	27.0	10797.7	1.2
Deadman Creek Rd.	South Bank	S18,19,30, T14N, R43E	1219.2	Low	13.7	28.0	1079.6	1.5
				Lower Granite Reservoir				
CPRR	North Bank	S33,34, T14N, R43E S2, T13N, R43E	4267.2	High	481.3	10.0	37788.1	9.1
Test Section	North Bank	Wawawai Creek to Steptoe Creek	16254.4	High	1833.4	10 and 11	143951.2	5.7
BNRR	North Bank	Steptoe Creek to RM 138.4	16459.2	Low	185.9	11.0	14598.5	4.0

Table H2, continued. Potential Failure Areas Resulting from a Permanent Drawdown

Feature	Location	Legal Description	Potential Failure Segment (m)	Class	Estimated Failure Length (m)	Mat. So. No.	Cubic Meters Required	Haul (kilometers)
Whitman Co. Rd. 9000	North Bank	Steptoe Creek to RM 138.4	11582.4	High	1306.4	11.0	102571.9	6.4
Hwy 12	South Bank	Alpowa Creek to Red Wolf Bridge	10972.8	Low	123.7	29 and 30	9716.5	5.2
Hwy 129	West Bank	RM 140.5 to 143	5486.4	High	618.7	32.0	48581.9	5.2
Nez Perce Co. Rd.	East Bank	Hwy 12 to RM 143	5486.4	Low	62.2	31.0	4882.0	3.3

While holding other parameters constant, the slope and height of the phreatic surface was varied according to the limits expressed above. Slopes range from 10 percent to 50 percent and are shown in radians. The phreatic surface ranges from 0.0 m to 4.5 m (anticipated ground surface) above the bedrock surface. The resulting factors of safety are shown in Table H3. The data shown indicate that, at slopes greater than about 30%, the factor of safety drops below 1 when the phreatic surface remains at the ground surface. Typical rates of permeability for silts and sandy silt mixtures (3.5×10^{-5} m^3/s or less) show that the phreatic surface would remain at the ground surface for a reservoir lowering rate of 2 feet per day, creating conditions of slope instability for slopes greater than 30 percent. For slopes of 40 percent and 50 percent, the instability would be much greater.

The study team devised a typical anticipated small failure from the observed data of the 1992 drawdown and a theoretical model based on natural slope instability. The following parameters were used:

- Length: 25.9 m
- Width: 3.7 m
- Depth: 1.5 m.

A cross section of the anticipated typical failure is shown in Figure H1. The quantities of construction materials for repair were calculated for the model using typical cross sections developed for the relocation of the County Road 9000 and the Camas Prairie Railroad. The quantities of the repair materials were then calculated for all projected small failures along the Snake River by multiplying the unit quantities (cubic meters per meter) by the number of feet of projected failure (also shown in Figure H1).

Figure H2 shows the cross section of a hypothetical large failure. The failure criteria, dimensions, and associated construction material quantities are also shown in Figure H2. It is anticipated that there would be at least two large failures on both the Little Goose and Lower Granite reservoirs, and one large failure on both the Ice Harbor and Lower Monumental reservoirs.

H.6 Conclusions

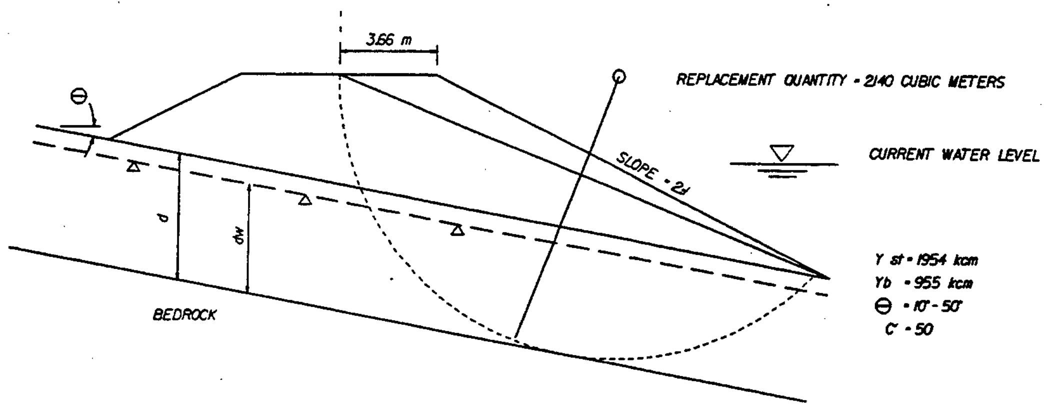
Drawdown would cause significant damage to road and railroad embankments. Most embankment failures are expected to occur after the reservoirs are significantly drawn down, when the excess weight of the water in the embankment materials would cause a failure. Temporary road detours may be required during and after drawdown to allow traffic to use roadways. However, railroad embankment failures may result in a shut down of rail traffic until repairs can be made. Rapid response approach to railroad repairs will be critical to minimizing the impacts of interruption of rail service.

H.7 Construction Schedule

Embankment repairs cannot be performed until after drawdown is accomplished. Also, in some areas, it may be necessary to wait several weeks after drawdown to allow the materials to drain and stabilize before repairs can be initiated. The exact number and extent of failures cannot be predicted prior to drawdown. Therefore, multiple equipment rental contracts would be awarded prior to drawdown, allowing repairs to be performed as failures occur. It is anticipated that most repairs would be completed within a few months and up to 1 year after drawdown is complete.

Table H3. Factors of Safety for Slope Stability

Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety	Degree Slope	Saturated Material Thickness (m)	Factor of Safety
5.7	0.0	6.11	11.3	0.0	3.06	16.7	0.0	2.04	21.8	0.0	1.54	26.6	0.0	1.23
5.7	0.3	5.88	11.3	0.3	2.94	16.7	0.3	1.96	21.8	0.3	1.48	26.6	0.3	1.19
5.7	0.6	5.65	11.3	0.6	2.83	16.7	0.6	1.89	21.8	0.6	1.42	26.6	0.6	1.14
5.7	0.9	5.43	11.3	0.9	2.72	16.7	0.9	1.81	21.8	0.9	1.37	26.6	0.9	1.10
5.7	1.2	5.21	11.3	1.2	2.61	16.7	1.2	1.74	21.8	1.2	1.31	26.6	1.2	1.05
5.7	1.5	5.00	11.3	1.5	2.50	16.7	1.5	1.67	21.8	1.5	1.26	26.6	1.5	1.01
5.7	1.8	4.79	11.3	1.8	2.40	16.7	1.8	1.60	21.8	1.8	1.21	26.6	1.8	0.97
5.7	2.1	4.59	11.3	2.1	2.30	16.7	2.1	1.53	21.8	2.1	1.16	26.6	2.1	0.93
5.7	2.4	4.39	11.3	2.4	2.20	16.7	2.4	1.47	21.8	2.4	1.11	26.6	2.4	0.89
5.7	2.7	4.19	11.3	2.7	2.10	16.7	2.7	1.40	21.8	2.7	1.06	26.6	2.7	0.85
5.7	3.0	4.00	11.3	3.0	2.00	16.7	3.0	1.34	21.8	3.0	1.01	26.6	3.0	0.81
5.7	3.4	3.81	11.3	3.4	1.91	16.7	3.4	1.28	21.8	3.4	0.96	26.6	3.4	0.77
5.7	3.7	3.63	11.3	3.7	1.82	16.7	3.7	1.22	21.8	3.7	0.92	26.6	3.7	0.74
5.7	4.0	3.45	11.3	4.0	1.73	16.7	4.0	1.16	21.8	4.0	0.87	26.6	4.0	0.70
5.7	4.3	3.28	11.3	4.3	1.64	16.7	4.3	1.10	21.8	4.3	0.83	26.6	4.3	0.67
5.7	4.6	3.10	11.3	4.6	1.55	16.7	4.6	1.04	21.8	4.6	0.78	26.6	4.6	0.63



ASSUMED TYPICAL FAILURE SECTION

NOT TO SCALE

DESIGN CRITERIA AND TYPICAL SMALL FAILURE PARAMETERS

I. SLIDE CHARACTERISTICS
   - A. LENGTH = 25.9 METERS
   - B. WIDTH = 3.7 METERS
   - C. DEPTH (VERTICAL DISPLACEMENT) = 1.5 METERS

II. EXCAVATION
   - A. QUANTITY OF MATERIAL ABOVE FAILURE ARC = 2140 CU. METERS

III. AVERAGE HAUL DISTANCES
   - A. ICE HARBOR POOL = 11.3 KILOMETERS
   - B. LOWER MONUMENTAL POOL = 11.7 KILOMETERS
   - C. LITTLE GOOSE POOL = 12.9 KILOMETERS
   - D. LOWER GRANITE POOL = 12.9 KILOMETERS

IV. MATERIAL TYPES AND QUANTITIES FOR SINGLE FAILURE
   - A. ROAD (8.5 METERS WIDE)
     - 1. SURFACING, COLD MIX ASPHALT = 7.3 CUBIC METERS
     - 2. SURFACE COURSE = 14.4 CUBIC METERS
     - 3. BASE COURSE = 11.5 CUBIC METERS
     - 4. FILL = 2090 CUBIC METERS
       - a. USE IN-PLACE MATERIAL. EXCAVATE AND RECOMPACT IN .3048 METERS LIFTS TO 95%
   - B. RAIL ROAD (5.5 METERS WIDE)
     - 1. BALLAST = 100.9 CUBIC METERS
     - 2. SUBBALLAST = 43.4 CUBIC METERS
     - 3. FILL = 1996 CUBIC METERS
       - b. USE IN-PLACE MATERIAL. EXCAVATE AND RECOMPACT IN .3048 METERS LIFTS TO 95%

V. MATERIAL TYPES AND QUANTITIES FOR ALL ANTICIPATED SMALL FAILURES (CUBIC METERS)

MATERIAL	ICE HARBOR	MONUMENTAL	LITTLE GOOSE	LOWER GRANITE
ASPHALT SURFACING	26	26	51	51
SURFACE COURSE	51	51	102	102
BASE COURSE	102	102	203	203
ROAD FILL	91,752	91,752	91,752	91,752
BALLAST	356	356	713	713
SUBBALAST	153	153	306	306
RAIL ROAD FILL	91,752	91,752	91,752	91,752

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

RAILROAD AND ROADWAY REPAIR
SMALL SLOPE FAILURES

Figure: H1

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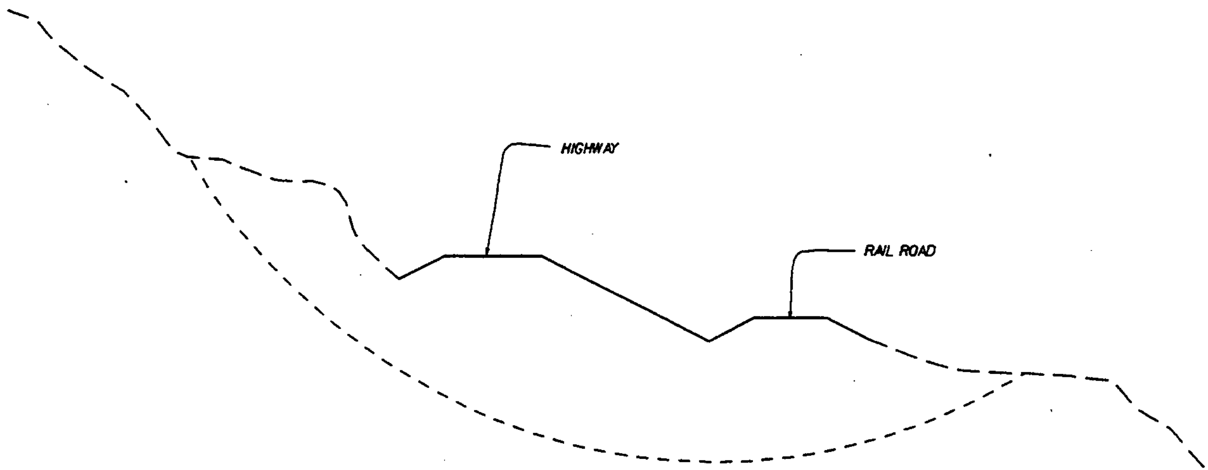

HYPOTHETICAL LARGE SCALE FAILURETYPICAL SECTION

NOT TO SCALE

DESIGN CRITERIA AND TYPICAL LARGE FAILURE PARAMETERS

I. SLIDE CHARACTERISTICS
- A. LENGTH = 91 METERS
- B. WIDTH = 91 METERS
- C. DEPTH = 46 METERS

II. ESTIMATED QUANTITY OF MATERIAL ABOVE FAILURE ARC
- A. 91,752 CU. METERS

III. AVERAGE HAUL DISTANCES
- A. ICE HARBOR POOL = 11.3 KILOMETERS
- B. LOWER MONUMENTAL POOL = 11.3 KILOMETERS
- C. LITTLE GOOSE POOL = 12.9 KILOMETERS
- D. LOWER GRANITE POOL = 12.9 KILOMETERS

IV. MATERIAL TYPES AND QUANTITIES FOR ALL ANTICIPATED LARGE FAILURES (CUBIC METERS)

MATERIAL	ICE HARBOR	MONUMENTAL	LITTLE GOOSE	LOWER GRANITE
ASPHALT SURFACING	560	385	911	1,293
SURFACE COURSE	1,107	763	1,802	2,558
BASE COURSE	2,220	2,243	3,604	5,131
ROAD FILL	160,962	162,634	261,302	372,094
BALLAST	7,771	7,852	12,616	17,965
SUBBALAST	3,338	3,373	5,420	7,717
RAIL ROAD FILL	159,721	155,318	2,459,240	355,353

US Army Corps of Engineers Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LARGE SLOPE FAILURES

Figure: H2

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SHEET MAX SCALE

Annex I

Lyons Ferry Hatchery Modification Plan

Annex I: Lyons Ferry Hatchery Modification Plan

I.1 General

The purpose of this annex is to discuss the modifications required for the Lyons Ferry Hatchery as a result of drawdown of the four lower Snake River dams. It is assumed that funding would be appropriated to modify Lyons Ferry Hatchery and water supply for operation during and after drawdown. All functions of the hatchery that would be affected by the drawdown were reviewed. The affected hatchery features include the water supply, collection of fish for spawning, return of selected fish to the river during the sorting process, release of smolts to the river at completion of rearing, and draining of hatchery process water into the Snake River.

In preparing this report, the study team reviewed original hatchery contract drawings to determine features likely to be affected by drawdown, consulted various designers knowledgeable about the Lyons Ferry Hatchery, and visited the site to discuss issues with the Hatchery Manager and gain a better understanding of hatchery operations. After reviewing the information, the team considered various options for each item that would be affected by the drawdown and selected a recommended approach for each item. The recommended approaches are conceptual plans intended to provide sufficient detail for a cost estimate.

I.2 Overview of Lyons Ferry Hatchery

The Lyons Ferry Hatchery was constructed as a part of the *Lower Snake River Fish and Wildlife Compensation Plan.* This plan consisted of construction of numerous fish facilities and development of habitat lands that were to serve as mitigation for fish and wildlife habitat lost or altered by construction of the four lower Snake River dams. Fish raised at Lyons Ferry Hatchery include steelhead, chinook salmon, and trout for release into the Snake River and its tributaries. Fish are also raised for research performed by the National Marine Fisheries Service, the U.S. Fish and Wildlife Service, and the Nez Perce Tribe.

I.3 Hatchery Features Requiring Modifications

The hatchery features that will require modifications due to drawdown are shown in Figures I1 and I2 and include the following:

1) Water wells No. 1 and 3 through 9 at Marmes, which provide all hatchery process water
2) Water well No. 2, which provides water for domestic and fire water
3) A 1,524-millimeter (mm)(60-inch) concrete cylinder pipe (CCP) for water supply from sta 9+88 to sta 72+51
4) Fish ladder and 1,372-mm (54-inch) main hatchery drain pipe
5) Steelhead exit channel
6) Two 305-mm (12-inch) fish return pipes from the steelhead spawning facility
7) A 610-mm (24-inch) corrugated metal pipe (CMP) drain from the pollution abatement pond.

For this feasibility study, the study team assumed that each of these facilities would need to remain operational during and following drawdown, or a substitute function would need to be provided. Minor outages of a day or two in the water supplies would be acceptable, if required, to tie new facilities into the existing ones.

Options for maintaining the function of each feature during and after drawdown were considered by the study team on the basis of the option's functional effectiveness, construction details, logistics, and

schedule requirements. The options were based on the assumption that drawdown would begin on the first of August and would take approximately 75 days to complete. Variations to some alternatives could be considered if drawdown were to occur during a different time of the year. For each feature modification, the study team examined the option of doing nothing, but concluded that in no instance would the "do nothing" option assure continuous hatchery operation, so this option was discarded.

I.3.1 Water Wells at Marmes

Description

A total of eight wells located at the Marmes site supply all process water for hatchery operations. All eight wells are located within a range of 37 meters (m) to 82 m (120 feet to 270 feet) from the edge of the reservoir and are drilled through Spokane flood sands and gravels. Water is pumped by vertical turbine pumps to a surge tank that provides head for gravity flow of the water to the hatchery. Salient characteristics of the wells are shown in Table I1.

Table I1. Well Characteristics

Well Number	Depth Below Existing Grade (feet)	Depth Below Reservoir Elevation, 540 feet	Flow (gallons per minute @ total dynamic head)	Pump HP	Column Connection Diameter (inches)
1	312	242	2600@75	75	12
3	343.5	276	6500@75	200	16
4	322.7	245	6500@75	200	20
5	350.7	303	6500@75	200	20
6	349.6	300.2	6500@75	200	20
7	311.1	260	920@75	300	20
8	311.4	260	920@75	300	20
9	309.3	260	920@75	300	20

When the reservoir is drawn down, the drop in water surface will affect the wells at the Marmes site. The exact nature and extent of the effect cannot be determined until drawdown has occurred and the water table around the wells stabilizes. The best case would be that the existing wells would still function, but at reduced capacity. The worst case would be that the water table would be drawn down below the water producing strata that would render the wells nonfunctional.

Options Considered

Option 1 - Drill New Wells to Replace/Supplement the Existing Wells Prior to Drawdown

Under this option, the existing wells would remain a part of the system and continue to be used as long as they functioned properly. A new set of wells of similar design and capacity, except drilled approximately 100 feet deeper, would be added in the immediate vicinity of the existing wells. The new wells would be tied into the existing water supply pipeline for delivery to the hatchery. The new wells would be designed and constructed prior to drawdown in an attempt to maintain sufficient water supply to the hatchery during and after the drawdown.

Option 2 - Drill New Wells to Replace/Supplement the Existing Wells After Drawdown

Under this option, the existing wells would remain a part of the system and continue to be used as long as they functioned properly. Following drawdown, a new set of wells of similar design with capacity

determined by need would be constructed after the water table had time to stabilize. (The study team assumed 1-year delay for water table stabilization.) The new wells would be tied into the existing water supply pipeline for delivery to the hatchery. The hatchery would operate on water from the existing wells during and after drawdown until the new wells were constructed. Hatchery operations would have to be modified, or completely terminated, depending upon the quantity of water produced by the existing wells. Until the new wells were installed, the hatchery would not have a definite, dependable amount of water for rearing fish.

Option 3 - Construct a River Intake with Pumping and Water Treatment Facilities

Under this option, a new river intake would be designed and built to provide the hatchery's water supply. The intake would be located on the Snake River upstream of all the hatchery drains and outfalls, a distance of approximately 150 m (500 feet) from the hatchery site. The intake would be a cast-in-place, reinforced concrete structure with a screened intake to provide up to 3.4 cubic meters per second (m^3/s) (120 cubic feet per second [cfs]) of river water to the hatchery. Water would be pumped to the hatchery, where it would be treated for temperature, dissolved gasses, turbidity, suspended solids, and pH. Construction of the river intake, pumping plant, and supply piping would not be practical prior to drawdown. A temporary means of water supply for the hatchery, considering the quality and quantity required, is not considered practical.

Comparison of Options and Recommendation

None of the previous options would provide a guaranteed supply of water of adequate quality and quantity to maintain continuous operation of Lyons Ferry Hatchery. Option 1 would attempt to provide a continuous water supply, but there is no assurance that the new system of wells would function properly after drawdown since their design would be based on, at best, predictions concerning the water table level after drawdown, which may, or may not, in fact be true once the water table re-stabilizes. Options 2 and 3 would provide greater assurance of a dependable water supply, but neither would be available until after the drawdown and construction of new facilities. Option 2 also would require time for the water table in the vicinity of the wells to adjust, so the new wells could be properly designed. Option 3 would be the surest source of water but not without problems. The facilities required to treat the river water would be expensive to design, construct, operate, and maintain. The intake and pumping facilities also would have the added risk of having to survive flood flows on the Snake River.

Future design activities could include a groundwater investigation with numerical modeling to provide a better estimate of the extent of additional wells that may be required. The need for such detailed analyses will depend on the operational options that are available to the hatchery for the time period where a reduced supply of water may result from reservoir drawdown. Those options may include the use of satellite capture facilities for the interim time period or as a long-term operation change.

Under the current operating plan, the water supply is fundamental to hatchery operation and cannot be shut down for extended periods during fish rearing. None of the options discussed above can guarantee a definite, dependable, good quality water supply both during and after reservoir drawdown. For the purposes of this feasibility study, Option 1 is recommended because it is the only option that attempts to maintain the water supply continuously. The hatchery can continue to use the existing wells at Marmes, and a new set of wells would be installed to provide additional capacity. However, the quantity of water provided by this option would be unknown until the drawdown has been completed and the water table stabilizes.

I.3.2 Water Well No. 2

Description

Water well No. 2 provides domestic and fire water for the hatchery. It is located approximately 200 m (656 feet) to the east of the hatchery near the Joso bridge. The well is drilled through Spokane flood sands and gravels and is 62 m (205 feet) deep. Water is pumped by submersible turbine pump to the hatchery site. A total of four pumps are in the well. The primary pump is a submersible turbine pump with a capacity of 2.5 cubic meters per minute m^3/m (650 gallons per minute [gpm]) at 23 m (75 feet) of total dynamic head (TDH) and a 127-mm (5-inch) diameter discharge. An identical backup pump is located in the well. Another submersible turbine pump has a capacity of 0.3 (80 gpm) at 53 m (175 feet) TDH and a 51-mm (2-inch) discharge. This pump also has an identical backup pump in the well.

When the reservoir is drawn down, the drop in water surface will affect well No. 2. The exact nature and extent of the effect cannot be determined until after the drawdown is completed and the water table around the well stabilizes. The best case would be that the existing well would still function, but at reduced capacity. The worst case is that the well would be rendered non-functional.

Options Considered

Option 1 - Install a New Well after Drawdown

Under this option, the existing well would be used during and after drawdown. If the quantity of water from the well dropped below that required to operate the fire protection system, a temporary pumping system would be furnished which could provide fire protection until a new well could be developed. If the drawdown resulted in a reduction of well capacity below that required for domestic use, a temporary supply of potable water would be furnished to the hatchery until a new well was developed. A new well would be constructed following drawdown, after allowing a sufficient time for the water table in the well vicinity to stabilize (the study team assumed 1 year after drawdown). This option would allow the hatchery to continue to operate with only minor inconvenience and would allow a new well to be developed at the most appropriate time (when the water table is stabilized after drawdown). If the well became non-functional, it would cause some inconvenience to the hatchery and would increase risk in the event of a fire.

Option 2 - Install a New Well Before Drawdown

Under this option, a new, deeper well would be installed in the vicinity of the existing well. The new well would be drilled 100 feet deeper in an attempt to maintain the existing well capacity following drawdown. This option would allow the hatchery to continue to operate without disruption of its domestic and fire water systems. It would, however, also run the risk that the new well would not provide adequate capacity following drawdown. The effect of the drawdown on the well cannot be accurately predicted prior to the drawdown.

Comparison of Options and Recommendation

Option 1 would likely cause some inconvenience to the hatchery during drawdown, but would allow the best well design. Option 2 would provide an adequate quantity of water for domestic and fire purposes, but only if the assumed increased well depth were adequate. Since the domestic and fire water can be provided by temporary means, and the existing well is likely to continue to operate at some capacity, Option 1 is the recommended option. It will allow the hatchery to continue operations during the drawdown and give the best long-term solution to domestic and fire water supply.

I.3.3 Water Supply Pipeline

Description

The main hatchery water supply line is a 1,524-mm (60-inch) diameter CCP that runs from the Marmes well site to the hatchery, a distance of approximately 2,966 m (9730 feet) (see Figure I1). The pipe is underground from its start at station 4+20 to station 9+50.49. From station 9+88 to station 72+76, the pipe is submerged in the reservoir and is supported by 104 pipe pile bents (see Figure I3). The bents are located at 20 m (64 feet) on center for all straight runs, with closer spacing at turning points.

The 1,524-mm (60-inch) CCP was designed to be supported by bents spaced at 20 m (64 feet) on center with the pipeline submerged. If the reservoir is drawn down, the pipe and supports will no longer be submerged. Analysis of the non-submerged pipe and support bents indicates the CCP will not have adequate flexural strength to span 64 m (64 feet) when full of water, and the pipe pile bents will not have adequate strength to resist seismic and wind loads.

Options Considered

Option 1 - Construct a New Pipeline With Adequate Structural Supports

Under this option, a complete new pipeline from station 9+88 to station 72+76 would be constructed. The pipeline would be similar to the existing one, but with an adequate support structure. A completely new pipeline would supply all hatchery water requirements plus, being new, would have the advantage of extending the design life of the system. This system could be constructed prior to drawdown by working from a floating plant, thereby maintaining hatchery function except for a short period required to tie into the existing system. A completely new pipeline, however, would be the most costly alternative and would take the longest to construct.

Option 2 - Construct a New Underground Pipeline

Under this option, a complete new pipeline from station 9+88 to station 72+76 would be constructed. The pipe would be similar to the existing one but would be buried in the ground in a manner similar to that used for the buried portions of the existing piping, instead of being supported on pile bents. Such a new pipeline also would supply all hatchery water requirements plus, being new, would have the advantage of extending the design life of the system. Because of the basalt cliffs near the Marmes site, however, the only practical alignment for an underground pipe is along the route of the existing water supply pipe (i.e. within the limits of the reservoir). Trenching along this route for installation would require waiting until the reservoir is drawn down and the reservoir bottom dried out sufficiently for construction equipment. The hatchery would have to be shut down until the new water supply pipe could be completed.

Option 3 - Construct a New Set of Piling Bents to Support the Existing Water Supply Pipeline

Under this option, a new set of pipe pile bents would be built to adequately support the existing water supply pipe. The bents would be similar in design and construction to the existing supports (see Figure I3). A new bent would be located at the center of each 20-m (64-foot) span between station 9+88 and station 72+76, for a total of 97 new bents. A new set of bents would make the existing pipeline structurally adequate without shutting down the water supply to the hatchery. The new bents could be installed from a floating plant prior to drawdown. This option has no apparent technical disadvantages. Although less costly than building a new water supply pipe with support structure, this would still be a relatively costly option.

Comparison of Options and Recommendation

Option 2 would not meet the requirement to maintain hatchery operation during and after the drawdown; therefore, only Options 1 and 3 can be considered further. Option 1, constructing a new pipe with adequate structural supports, would maintain hatchery operation, but would also be the most costly alternative. It would also require shutting down the water supply system to tie in the new pipe to the existing system near station 9+88 and station 72+76. Option 3, adding a new set of piling bents, would completely satisfy the requirement to maintain hatchery operation and would also be significantly less costly than Option 1. Therefore, Option 3 is the recommended option.

It should be noted, however, that the existing 1,524-mm (60-inch) CCP may be nearing the end of its service life when drawdown occurs. The decision to replace the pipe may be the best decision at that time. The replacement might use the existing supports along with additional supports as required, or might require a complete new set of supports.

I.3.4 Fish Ladder and Hatchery Drain

Description

The fish ladder is a cast-in-place, reinforced concrete structure located along the reservoir edge at the south end of the hatchery complex (see Figure I2). Flows of between 76 m^3/m (20,000 gpm) and 198 m^3/m (51,000 gpm) are released through a 1,372-mm (54-inch) drain pipe at the downstream end of the ladder to provide attraction water for upstream migrating adult salmon and steelhead. Except for minor amounts of water used for fish release, all process water from the hatchery is released through this drain. A 26-m^3/m (6,900-gpm) pump located adjacent to the ladder removes water from the drain pipe at a point approximately 30 m (100 feet) upstream of the outlet. This water is pumped into the upper diffuser of the fish ladder to provide flows through the ladder for fish passage. The fish ladder is operated during the period of July 1 through November 15 for collection of steelhead, and between September 1 and December 15 for collection of fall chinook salmon.

The fish ladder is designed to operate at reservoir elevations in the range of approximately 164 m to 165 m (537 feet to 540 feet) mean sea level (msl). (Mean sea level refers to North American Vertical Datum of 1929.) If the reservoir is drawn down, the fish ladder will not function and the drain will empty onto the bank and flow overland to the river. The bank of the river will be approximately 152 m (500 feet) horizontally from the ladder. Also, the free flowing river will range in elevation from approximately 143 m (468 feet) msl at 280 m^3/s (10,000 cfs) to approximately 147 m (482 feet) msl at approximately 2,266 m^3/s (80,000 cfs) during the period of time the ladder is operated each year.

Options Considered

Option 1 - Build a Ladder and Holding Pond on the River Bank with Access Road to the Hatchery

This option would include construction of a cast-in-place, reinforced concrete ladder similar to the existing ladder at the edge of the river along with a holding pond with crowder and loading arrangement. An access road would be constructed between the hatchery and the new ladder/trap. Also, a fish hauling truck to carry fish back to the hatchery facility would be required. This option would require that the 1,372-mm (54-inch) drain be routed to the new facility to be used for attraction water and a new pumping arrangement be designed to provide flows in the ladder. Also, the existing ladder and entrapment structure would require modification to maintain the hydraulic function, since flows from the entrapment structure pass out through the ladder. Because the new ladder and holding pond, as well as a portion of the access road, would be built in the flood way, the facilities below elevation 151 m (495 feet) msl would

require design features to prevent damage during flood events. Construction of the ladder extension and drain extension would not be practical until the reservoir was drawn down and the bank sufficiently dried out to allow construction activities. During drawdown and until the new facilities were complete, water from the drain would have to be routed to the river in a manner to prevent erosion of the bank and high sediment discharge into the river. An alternate source of eggs to maintain hatchery operation would be required for the fish season during the year of drawdown. Eggs would be provided from fish trapped at Ice Harbor Dam or possibly from fish trapped at satellite hatcheries, such as the Tucannon Hatchery. This is a complicated alternative with not only significant new construction required, but also modification of the hatchery's operating procedures to include collecting fish at a new location and hauling them to the hatchery for spawning.

Option 2 - Extend the Existing Fish Ladder and Drain to the River Bank

This option would involve extension of the existing ladder to the river bank along with the 1,372-mm (54-inch) drain. The ladder extension would be a cast-in-place, reinforced concrete structure similar to the existing ladder (see Figures I4 and I5). The drain would run parallel to the ladder extension to the river bank and its outfall would be at the downstream end of the ladder extension to provide fish attraction water. The existing 26-m^3/m (6,900-gpm) pump, which provides water to the upper diffuser, would be adequate for fish passage. The ladder extension would require an entrance configuration that would be operational for water surface elevations from 143 m (468 feet) msl at 283 m^3/s (10,000 cfs) to 147 m (482 feet) msl at 2,266 m^3/s (80,000 cfs). Also, since the ladder extension would be constructed into the floodway, the structure below elevation 157 m (495 feet) msl would be designed to prevent damage during flood events. Construction of the ladder extension and drain extension would not be practical until the reservoir was drawn down and the bank sufficiently dried out to allow construction equipment to operate. During drawdown and until the new facilities were complete, water from the drain would have to be routed to the river in a manner to prevent erosion of the bank and high sediment discharge into the river. An alternate source of eggs to maintain hatchery operation would be required for the fish season during the year of drawdown. Eggs would be provided from fish trapped at Ice Harbor Dam or possibly from fish trapped at satellite hatcheries, such as the Tucannon Hatchery.

Comparison of Options and Recommendation

Neither of these options would meet the requirement to maintain the operation of the fish ladder during the year of the drawdown.

Options 1 and 2 would require an alternate source of eggs for one fish rearing season either by collecting them at Ice Harbor Dam or from satellite hatcheries, such as Tucannon Hatchery. Option 2 is simpler than Option 1 in that it only extends the existing ladder and 1,372-mm (54-inch) drain pipe out to the bank of the river. It does not involve changes to spawning operations, the complication of trucking fish to the existing facilities for spawning, extra handling of fish, or construction of new pumping facilities to furnish water to a new facility. Both Options 1 and 2 maintain a supply of eggs to the hatchery after drawdown and construction of new facilities are completed. Option 2 would have less effect on hatchery operations and is anticipated to be less costly. Therefore, Option 2 is the recommended option.
The new fish ladder structure for either Option 1 or 2 would need to operate over a wide range of river levels. It would be a difficult design and could be a substantial and very costly structure.

It should also be noted that the 1,372-mm (54-inch) drain pipe would require temporary measures for diversion and care of water during the drawdown until the new facilities were constructed for either Option 1 or Option 2. One method would be to provide a riprap blanket to protect the bank from erosion. A blanket 3 m (10 feet) wide by 0.6 m (2 feet) thick by 152 m (500 feet) long should be adequate.

I.3.5 Steelhead Exit Channel

The steelhead exit channel is a cast-in-place, reinforced concrete channel that is the outlet channel from the steelhead collection structure (see Figure I2). Both steelhead and salmon, reared in the three large rearing ponds, are released through this channel into the reservoir at the end of the rearing cycle. A flow of 17 m^3/m (4,500 gpm) is released through the exit channel during the emptying of each large rearing pond. Fish and water flow from the channel outfall at invert elevation 165 m (540.75 feet) msl.

The steelhead exit channel was designed to operate at reservoir elevations in the range of approximately 164 m (537 feet) to 165 m (540 feet) msl. If the reservoir is drawn down, the river will be approximately 146 m (480 feet) horizontally from the channel outfall and the water surface will be as low as elevation 143 m (468 feet) msl. Water from the channel would flow overland to the river, causing unacceptable erosion and turbidity in the river. Fish mortality would likely be 100 percent.

Options Considered

Option 1 - Extend the Steelhead Exit Channel with a Cast-in-Place, Reinforced Concrete Channel

Under this option, the existing channel would be extended approximately 146 m (480 feet) to the edge of the river with a cast-in-place, reinforced concrete channel similar to the existing channel. The flow of 17 m^3/m (4,500 gpm) on a slope of 0.0158 gives a channel width of 457 mm (18 inch) with a normal depth of approximately 102 mm (4 inch) and a velocity of 6 m (20 feet) per second. The slope should be flattened out near the river to provide an impact velocity for fish entering the river of less than 9 m (30 feet) per second. The channel would be in the floodway below elevation 151 m (495 feet) and would be designed for appropriate flood flows by setting the channel flush with the existing groundline and providing rock armoring or other appropriate measures where necessary. This option would not be practical to construct prior to drawdown of the reservoir and, therefore, would require an interim plan for the drawdown year to allow for the banks to dry out and for construction of the channel extension. Fish from the rearing ponds could be pumped from the steelhead collection structure with a fish pump and transported to and released in the river with fish hauling trucks. Water from the steelhead exit channel would be diverted to the 1,372-mm (54-inch) main facility drain.

Option 2 - Extend the Steelhead Exit Channel with a High-Density Polyethylene (HDPE) Pipe

Under this option, the existing channel would be extended approximately 146 m (480 feet) to the edge of the river with a HDPE pipe. The flow of 17 m^3/m (4,500 gpm) on a slope of 0.0158 gives a pipe diameter of 610 mm (24 inch) with a normal depth of approximately 127 mm (5 inch) and a velocity of 6 m (21 feet) per second. The slope should be flattened out near the river to provide an impact velocity for fish entering the river of less than 9 m (30 feet) per second. The pipe would be buried in the ground except where it ties into the existing channel and at its outfall near elevation 143 m (468 feet) msl. The outfall would be designed for appropriate flood flows by providing rock armoring. This option would not be practical to construct prior to drawdown of the reservoir and, therefore, would require an interim plan for the drawdown year to allow for the banks to dry out and for construction of the channel extension. Fish from the rearing ponds could be pumped from the steelhead collection structure with a fish pump and transported to and released in the river with fish hauling trucks. Water from the steelhead exit channel would be diverted to the 1,372-mm (54-inch) main facility drain.

Comparison of Options and Recommendation

Both Options 1 and 2 would require a one-year modification to the hatchery's operation involving use of an alternate method of moving fish from the large rearing ponds to the reservoir. Both options also would

serve the hatchery's purpose. Option 1 is a more complicated type of construction than Option 2 and would likely cost significantly more. Option 1 also would be more difficult to design for flood flows than Option 2. On the basis of cost and simplicity of design, Option 2 is the recommended option.

It should also be noted that, if the assumption that hatchery operations would not be modified were relaxed, the option of abandoning the steelhead exit channel and providing fish release by pumping fish into fish hauling trucks for release into the river would be feasible.

I.3.6 Steelhead Spawning Fish Return Pipes

Description

The steelhead spawning fish return pipes are two 305-mm (12-inch) diameter polyvinyl chloride (PVC) pipes that are used to release fish from the steelhead spawning building. The pipes pass out the south foundation wall of the building, through the fish ladder, with the outfall for both pipes located upstream of the fish ladder entrance.

The steelhead spawning fish return pipes were designed to operate at reservoir elevations in the range of approximately 164 m (537 feet) to 165 m (540 feet) msl. If the reservoir is drawn down, the river will be approximately 152 m (500 feet) horizontally from the channel outfall, and the water surface will be as low as elevation 143 m (468 feet) msl. Water from the pipes would flow overland to the river, causing unacceptable erosion and turbidity in the river. Fish mortality would likely be 100 percent.

Options Considered - Extend the Steelhead Spawning Fish Return Pipes to the River

Only one feasible option was defined for this modification. Under this option, the fish return pipes would be extended approximately 152 m (500 feet) to the edge of the river with a 305-mm (12-inch) diameter HDPE pipe. The two 305-m (12-inch) PVC pipes would be merged into one 305-mm (12- inch) pipe that would extend to the river at elevation 143 m (468 feet) msl. The pipe would be buried in the ground except where it ties into the existing channel and at its outfall near elevation 143 m (468 feet) msl. The outfall would be designed for appropriate flood flows by providing rock armoring. This option would not be practical to construct prior to drawdown of the reservoir and, therefore, would require an interim plan for the drawdown year to allow for the banks to dry out and for construction of the pipe extension. Fish from the spawning building could be collected during the interim year and transported to and released in the river with fish hauling trucks.

This option would require a one-year interim plan to release fish from the steelhead spawning building. This does not satisfy the assumption that hatchery operations should not be affected, but otherwise is a relatively simple option that would maintain the function of the 305-mm (12-inch) PVC fish release pipes.

It should also be noted that, if the assumption that hatchery operations would not be modified were relaxed, the option of abandoning the 305-mm (12-inch) PVC fish release pipes and providing fish release by hauling fish to the river in trucks would be feasible.

I.3.7 Pollution Abatement Pond Drain

Description

The pollution abatement pond is an earthen pond that functions as a settling basin where water from hatchery cleaning operations is treated prior to being released into the reservoir. As designed, the

clarified water flows into the reservoir from a concrete outlet structure by flowing over an overflow weir constructed of wooden stoplogs, then flowing out through a 610-mm (24-inch) diameter CMP that passes through the earth berm which forms the south side of the pond. Only small intermittent flows from cleaning operations are sent to the pollution abatement pond. Consequently, little, if any, water actually flows into the reservoir.

The pollution abatement pond was designed to operate at reservoir elevations in the range of approximately 164 m (537 feet) to 165 m (540 feet) msl. If the reservoir is drawn down, the river edge would be approximately 122 m (400 feet) horizontally from the pollution abatement pond outfall. Water from the outfall pipe would flow overland toward the river causing unacceptable erosion and turbidity in the river.

Options Considered

Option 1 - Construct a Seepage Pit for the Pollution Abatement Pond Effluent

Under this option, a seepage pit would be constructed to allow effluent from the pollution abatement pond to seep into the soils adjacent to the hatchery facility. The pit would be located approximately 15 m (50 feet) to the south of the existing pond outfall. The seepage pit would be a 6-m (20-foot) diameter by 4.5-m (15-foot) deep circular pit with walls constructed of concrete circular type cesspool blocks or other approved materials. The pit would be covered with an arched, reinforced concrete lid with manhole cover. Coarse gravel 305-mm (1-foot) thick would be placed in the bottom of the pit. Backfill around the outside of the walls would consist of a 610-mm (2-foot) thick zone of 457 to 610 mm (1-1/2 to 2 inch) clean crushed stone. The existing 610-mm (24-inch) diameter CMP would be extended 15 m (50 feet) and would terminate in the seepage pit. This option would not be practical to construct prior to drawdown of the reservoir and, therefore, would require an interim plan for the drawdown year to allow for the banks to dry out and for construction of the seepage pit. For approximately six months following drawdown, the hatchery would pump effluent from the outfall structure to the 1,372-mm (54-inch) main facility drain during cleaning operations.

Option 2 - Extend the Pollution Abatement Pond Effluent Pipe to the River

Under this option, the existing 610-mm (24-inch) diameter CMP outlet would be transitioned down to a 305-mm (12-inch) diameter CMP that would be routed directly to the river. The pipe extension would be approximately 122 m (400 feet) long. A 305-mm (12-inch) CMP extending 122 m (400 feet) from elevation 166 m (545 feet) msl to 143 m (470 feet) msl could pass 0.3 m^3/s (9.1 cfs) flowing half full, which would be more than adequate. This option would not be practical to construct prior to drawdown of the reservoir and, therefore, would require an interim plan for the drawdown year to allow for the banks to dry out and for construction of the CMP extension. The outfall would be designed for appropriate flood flows by providing rock armoring. For approximately six months following drawdown, the hatchery would pump effluent from the outfall structure to the 1,372-mm (54- inch) main facility drain during cleaning operations.

Comparison of Options and Recommendation

Both Options 1 and 2 would satisfy the requirement to maintain the function of the pollution abatement pond outlet. Option 1 is more complicated and would require further analysis to verify that it is adequately sized. Option 2 is simpler construction than Option 1, would easily satisfy all flow requirements that might be expected from the pollution abatement pond, and is likely to be the least costly option. Therefore, Option 2 is the recommended option.

I.4 Conclusions And Recommendations

Drawdown would significantly affect hatchery operation. The hatchery depends on large quantities of ground water from wells drilled in alluvial sand and gravel. The existing wells may not provide the required volume of water. However, it is expected that there would be enough water for the hatchery to operate at a reduced rate until ground water availability can be evaluated after drawdown. If the hatchery is to operated without interruption, additional pipe bents must be constructed to support the water supply pipe before drawdown occurs. Therefore, it is recommended that the following modifications be completed prior to drawdown:

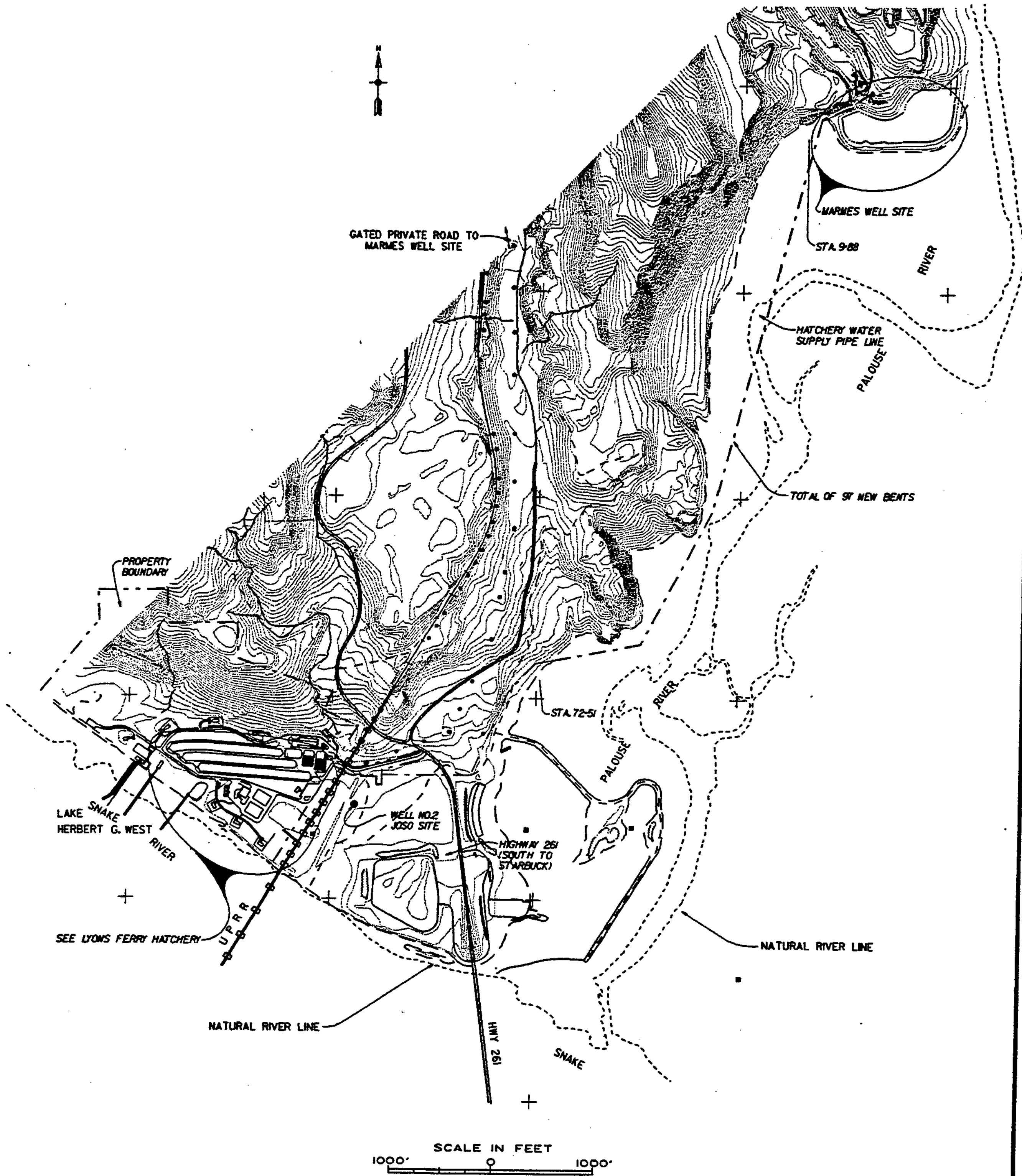
1) Install new pumps and set the intakes as deep as possible in the existing hatchery water supply wells and in the domestic well.
2) Install three new hatchery water supply wells to offset the expected reduced water production from the existing wells.
3) Install additional pipe pile bents be to support the existing water supply pipe.
4) Provide erosion protection along the slope from the 1,372-mm (54-inch) hatchery drain to the river.

The remaining modifications identified in this report would be performed after drawdown has occurred.

I.5 Construction Schedule

The construction schedule for the required hatchery modifications is as follows:

1) New pumps would be installed and the intakes set as deep as possible in the existing hatchery water supply wells and in the domestic well prior to drawdown.
2) Three new hatchery water supply wells would be constructed to augment the existing wells prior to drawdown.
3) Additional pipe pile bents would be constructed prior to drawdown to support the existing water supply pipeline between stations 72+76 and 9+88.
4) Erosion protection from the 1,372-mm (54-inch) hatchery drain to the river would be provided prior to drawdown.
5) The steelhead exit channel would be modified after drawdown.
6) A new domestic well would be drilled after drawdown.
7) The fish ladder would be modified after drawdown.
8) The steelhead spawning fish return pipes would be modified after drawdown.
9) The pollution abatement pond drain would be modified after drawdown.



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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LYONS FERRY HATCHERY VICINITY MAP

Figure: I1

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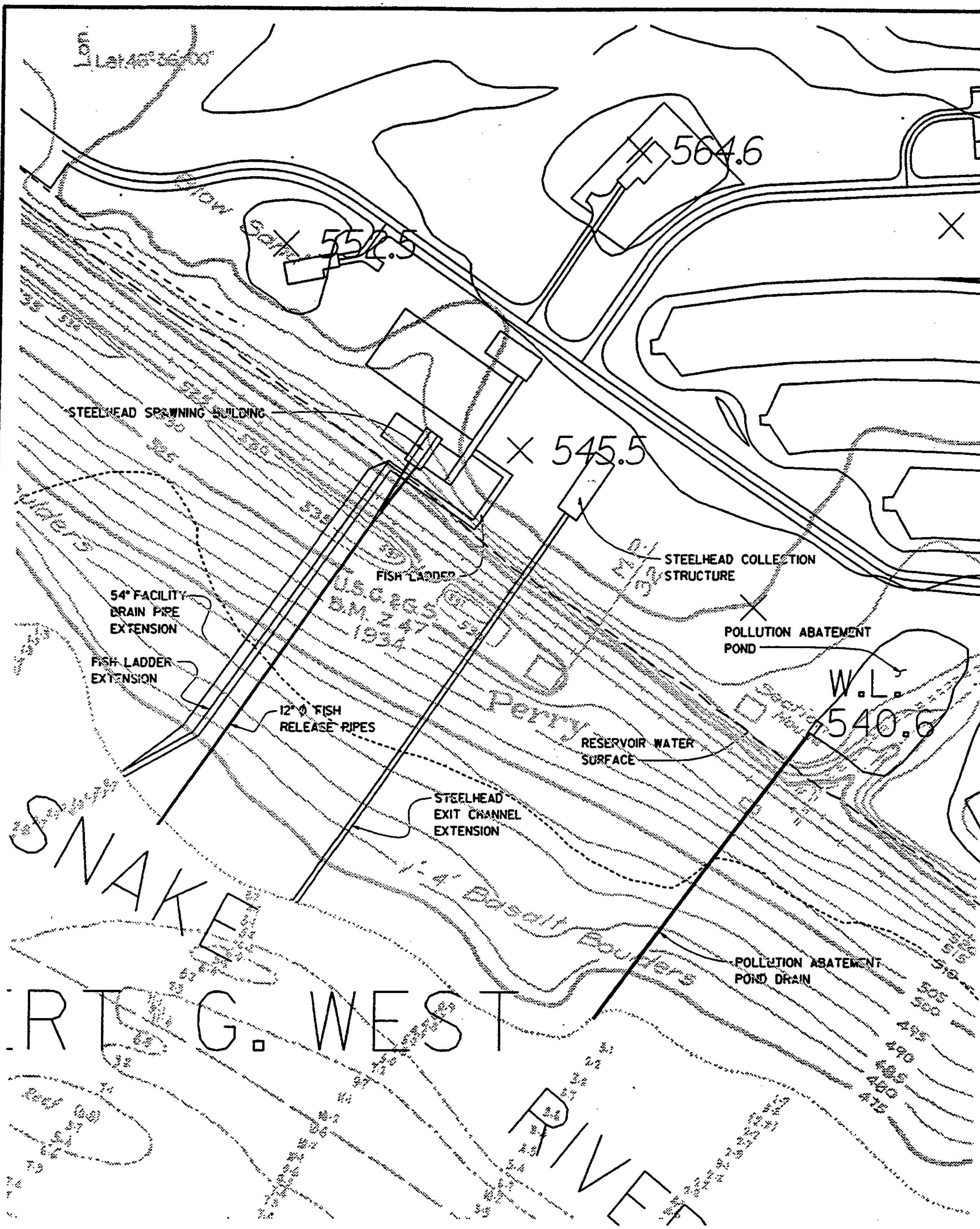

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Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

LYONS FERRY HATCHERY SITE PLAN

Figure: I2

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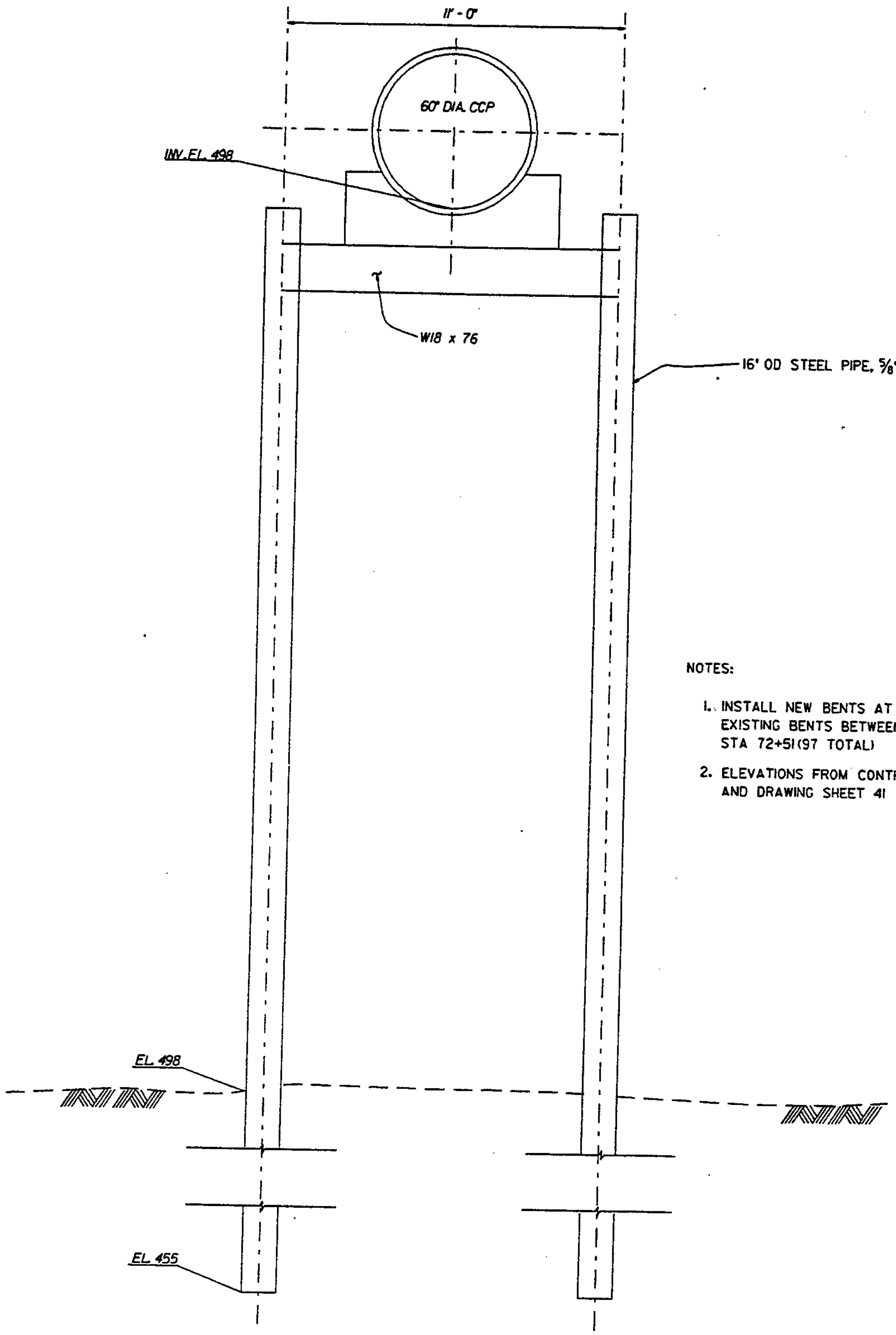

NOTES:

1. INSTALL NEW BENTS AT MIDSPAN OF EXISTING BENTS BETWEEN STA 9+88 AND STA 72+51 (97 TOTAL)
2. ELEVATIONS FROM CONTRACT 81-C-85 SPEC 2B AND DRAWING SHEET 41

WATER SUPPLY PIPE
STA. 65+47

SCALE IN FEET
12" 0 5'

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

WATER SUPPLY PIPE

Figure: I3

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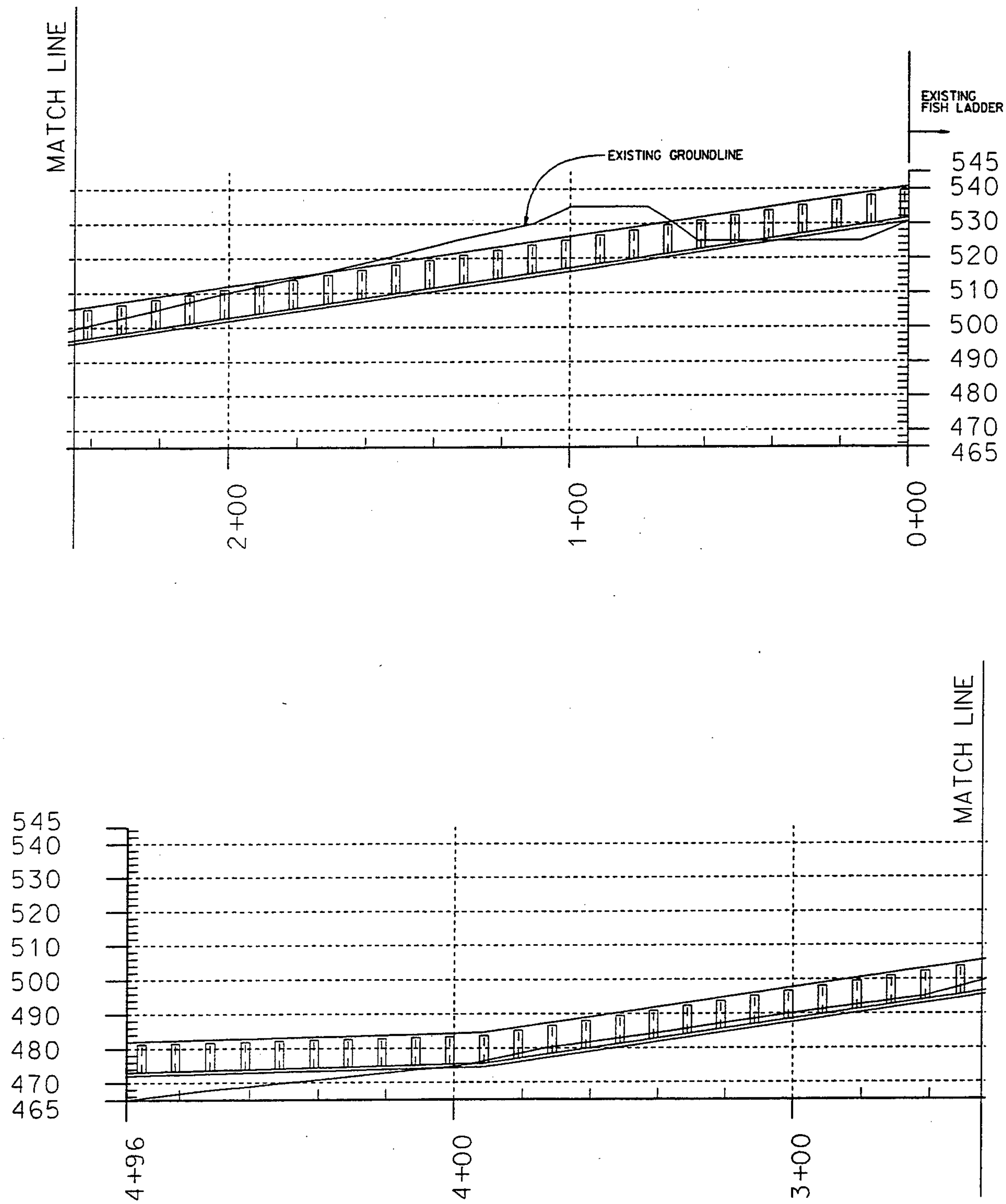

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

FISH LADDER EXTENSION PROFILE

Figure:
I4

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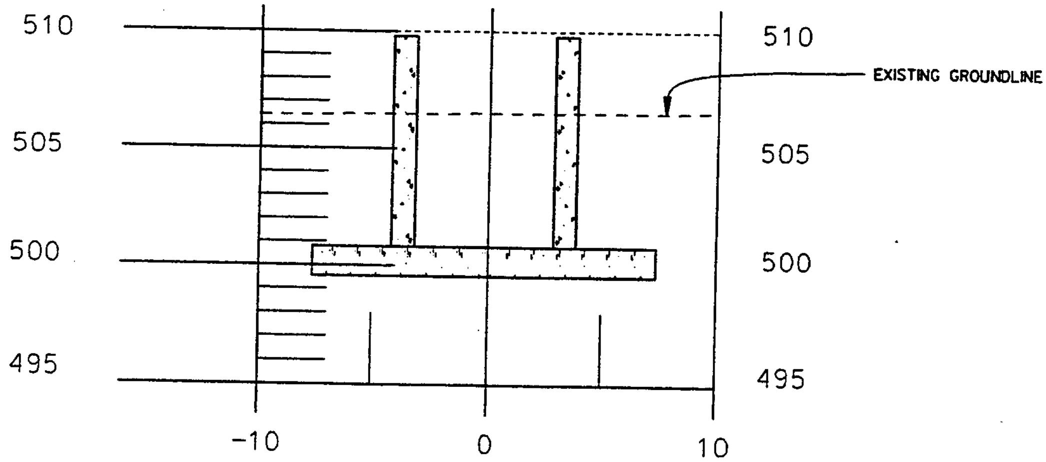
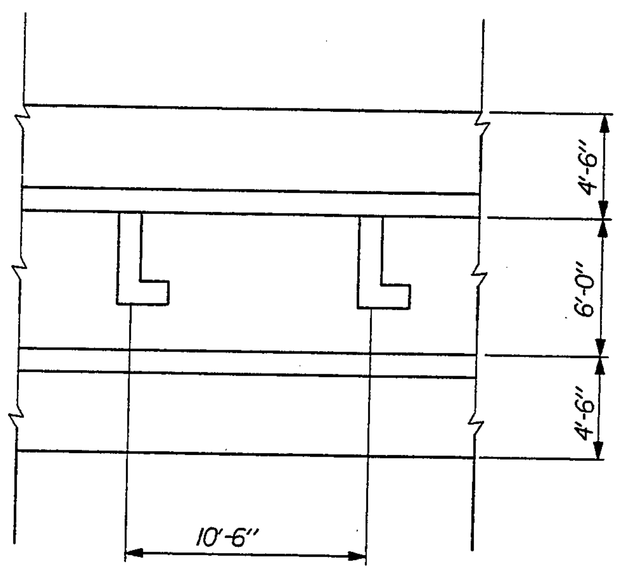


TYPICAL SECTION



TYPICAL PLAN

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

FISH LADDER EXTENSION DETAILS

Figure: I5

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Annex J

Habitat Management Units Modification Plan

Annex J: Habitat Management Units Modification Plan

J.1 General

Habitat Management Units (HMUs) were established to compensate for lost wildlife habitat due to reservoir impoundments behind the Snake River dams. Under the Lower Snake River Fish and Wildlife Compensation Plan, 30 areas were purchased and set aside for wildlife habitat mitigation. Most of the HMUs are non-irrigated; however, 10 HMUs have irrigation systems that are either supplied by surface water intakes in the river or by ground water wells.

After reservoir drawdown, the river corridor would begin to develop a natural habitat system. However, the system cannot be expected to rebound and stabilize immediately. Ultimately, fish and wildlife habitat and riparian zones would develop to a level that is self sustaining and renewing. Until riparian habitat can be re-established in the corridor, however, HMUs would continue to be maintained.

The location of the HMUs relative to the river would not be ideal after drawdown. In many cases, the HMUs would be significantly higher than the active water surface, and, in some cases, would be located a significant distance from the new river. The major impact of drawdown on the HMUs would be the disruption of the existing irrigation systems. The lower river water surface would render the river intake pumping systems inoperable and would significantly affect the water wells.

This annex presents the modifications needed to maintain water supplies for the irrigated HMUs. The information is presented primarily in tabular form for organization and ease of reference.

J.2 Methods

HMUs are classified as irrigated and nonirrigated. The irrigated HMUs include one or more wells or pumping stations for water supply. There are currently eight HMUs being irrigated by 11 surface-water pumping plants and two HMUs being irrigated by well-supplied water. Table J1 identifies which HMUs are presently irrigated and require water supply modifications.

Table J1. Irrigated HMUs Along the Snake River

HMU	Water Supply Source
Big Flat	River Intake, Pump Stations
Lost Island	River Intake, Pump Stations
Hollebeke	River Intake, Pump Station
Skookum	River Intake, Pump Station
Fifty-five Mile	River Intake, Pump Station
Ridpath	Ground Water Well
New York Bar	River Intake, Pump Station
Swift Bar	River Intake, Pump Station
John Henley	Ground Water Well
Chief Timothy	River Intake, Pump Stations

To determine the elevation and distance change of the HMUs relative to the river, they were all located on U.S. Geological Survey topographic maps, 7.5 minute, 1:24,000 scale. The resulting head differential after drawdown and the distance from the existing pump to the new river location were determined from the topographic maps. This information is summarized in Table J2.

Each pumping station would have to be modified to accommodate the lower and more fluctuating water surface levels. Figure J1 shows a typical existing water intake system. Table J3 summarizes the required piping and increased pump requirements for each pump station. Each new surface water pump intake would consist of a precast concrete headwall structure. The structure contains the intake and fish screen and provides a platform on which to set the pump motor and support for the pump and pipe column. Rock fill would be placed from shore to the intake in order to maintain permanent access to the pump and intake.

Since installation of new headwall structures cannot be done prior to drawdown, temporary measures must be implemented for the irrigation period of 1 August to approximately early October. Temporary measures include utilization of trailer-mounted pumps and flexible piping.

Depending on subsurface stratigraphy and the water surface change due to drawdown, the depth of each ground water well and the pump capacity may have to be increased to maintain a constant water supply. Table J4 presents all pertinent data on each of the water wells and summarizes well modification requirements. Table J5 shows the new pump requirements at each well.

J.3 Construction Schedule

It would not be practical to perform any of the HMU water supply construction modifications prior to drawdown. However, arrangements for electrical power extensions should be completed, and the surface water intake pumps and associated equipment could be purchased prior to drawdown. The two water wells would not be modified until drawdown is complete and the groundwater has had an opportunity to stabilize. It is possible that the wells might need to be drilled deeper than anticipated or additional wells might have to be drilled.

The drawdown period is scheduled to occur between August and December. It may be possible to begin construction of the permanent water intakes during the winter following drawdown. Considering the extent of sediment transport in the river during the first spring freshet, it is more prudent to continue use of the temporary irrigation equipment and begin construction during the summer following high river flows.

Table J2. Pump Station Modification Data

HMU	Feature	Type	Existing Pump Data HP	Existing Pump Data Capacity (liters/sec)	Existing Pump Data Head (m)	Relocation Data Added Head (m)	Relocation Data Distance (m)	Pumps Required	Notes
Big Flat	Pump Sta.	split case centrifugal		132.5	102.1	30.5	121.9	132.5 liters/sec @ 132.6 m	Two pumps are drawing from a single intake. Water rights data shows that 336 acres are being irrigated with up to 8.6 cfs or 672 acre-feet per year. The reservoir would be lowered about 30 m at the site and laterally about 122 m.
		split case centrifugal		111 liters/sec	93.0	30.5	121.9	111 liters/sec @ 123.4 m	The intake is a rectangular pattern with 30.48 mm (12-inch) slotted PVC with 50.8 mm (20-inch) galvanized steel pipe from intake to the pump.
Lost Island	Pump Sta.	split case centrifugal		132.5 liters/sec	102.1	21.3	91.4	50.5 liters/sec @ 85.3 m	Two pumps draw from a single intake. Water rights data shows that 81 acres are irrigated with up to 1.8 cfs or 162 acre-feet per year. The reservoir will be lowered about 21 m at the site and receded about 300 feet.
		split case centrifugal		111 liters/sec	93.0	21.3	91.4	50.5 liters/sec @ 85.3 m	The intake is a rectangular pattern with 30.48 mm (12-inch) slotted PVC with 50.8 mm (20-inch) galvanized steel pipe from intake to the pump.
Hollebeke	Pump Sta.	centrifugal		40.4 liters/sec	88.4	21.3	457.2	38.1 liters/sec @ 109.7 m	Single electric centrifugal pump irrigates 150 acres at a rate of 3.2 cfs and up to 244 acre feet per year.
Skookum	Pump Sta.	diesel		50.5 liters/sec	105.2	25.9	335.3	50.5 liters/sec @ 131.1 m	Single diesel engine driven pump supplies 75 acres at a rate of 1.8 cfs or 150 acre-feet per year.
Fifty-five Mile	Pump Sta.	turbine?		20.2 liters/sec	150.9	25.9	304.8	20.2 liters/sec @ 176.8	Single electric turbine pump supplies water to 29 acres at a rate of 0.71 cfs or up to 58 acre-feet per year.
New York Bar	Pump Sta.	centrifugal	125	60.6 liters/sec	99.1	25.9	457.2	60.6 liters/sec @ 99.1 m	Single electric centrifugal pump irrigates 98 acres at a rate of 2.13 cfs and up to 163 acre feet per year.
Swift Bar	Pump Sta.	turbine		70.6 liters/sec	85.3	12.2	182.9	70.6 liters/sec @ 85.3 m	Single electric turbine pump irrigates 104 acres at rate of 2.49 cfs up to 208 acre-feet per year.
Chief Timothy	Pump Sta., Pump #6	diesel	60	30.3 liters/sec	76.2	9.1	213.4	30.3 liters/sec @ 85.3 m	Single 60 hp 7.62 mm x 10.16 mm (3 inch x 4 inch) horizontal split case centrifugal pump irrigates 41.8 acres with up to 1.07 cfs.
	Pump Sta., Intake #1	turbine	40	31.5 liters/sec	73.2	12.2	609.6	31.5 liters/sec @ 85.3 m	Single 25.4 mm (10") submersible vertical turbine with 40 hp electric motor irrigates 14.5 acres with up to 1.1 cfs.

Table J3. Surface Water Intake Pump and Piping Modifications

HMU	Flow (gallons per minute)	Added head (m)	Added horizontal length (m)	Assumed pump effectiveness	Calculated new pipe size (mm)	Nominal new pipe size (mm)	New pipe length (m)	Pressure in new pipe (psi)	Calculated hp required new pump	Nominal hp required new pump
Big Flat	2100	30.5	122	0.75	30.5	30.5	126	55	89	100
Big Flat	1760	30.5	122	0.75	10.9	30.5	126	55	75	75
Lost Island	800	21	91	0.75	18.8	8.0	94	41	25	25
Lost Island	800	21	91	0.75	18.8	20.3	94	41	25	25
Hollebeke	640	21	457	0.75	18.8	20.3	458	49	24	25
Skookum	800	26	335	0.75	18.8	20.3	336	53	33	40
Fifty-five Mile	320	26	305	0.75	11.9	15.2	306	52	13	15
New York Bar	960	26	457	0.75	20.5	20.3	458	55	41	50
Swift Bar	1120	12	183	0.75	22.2	25.4	183	30	26	30
Chief Timothy	480	9	213	0.75	14.6	15.2	214	26	10	10
Chief Timothy	500	12	610	0.75	14.4	15.2	610	39	15	15

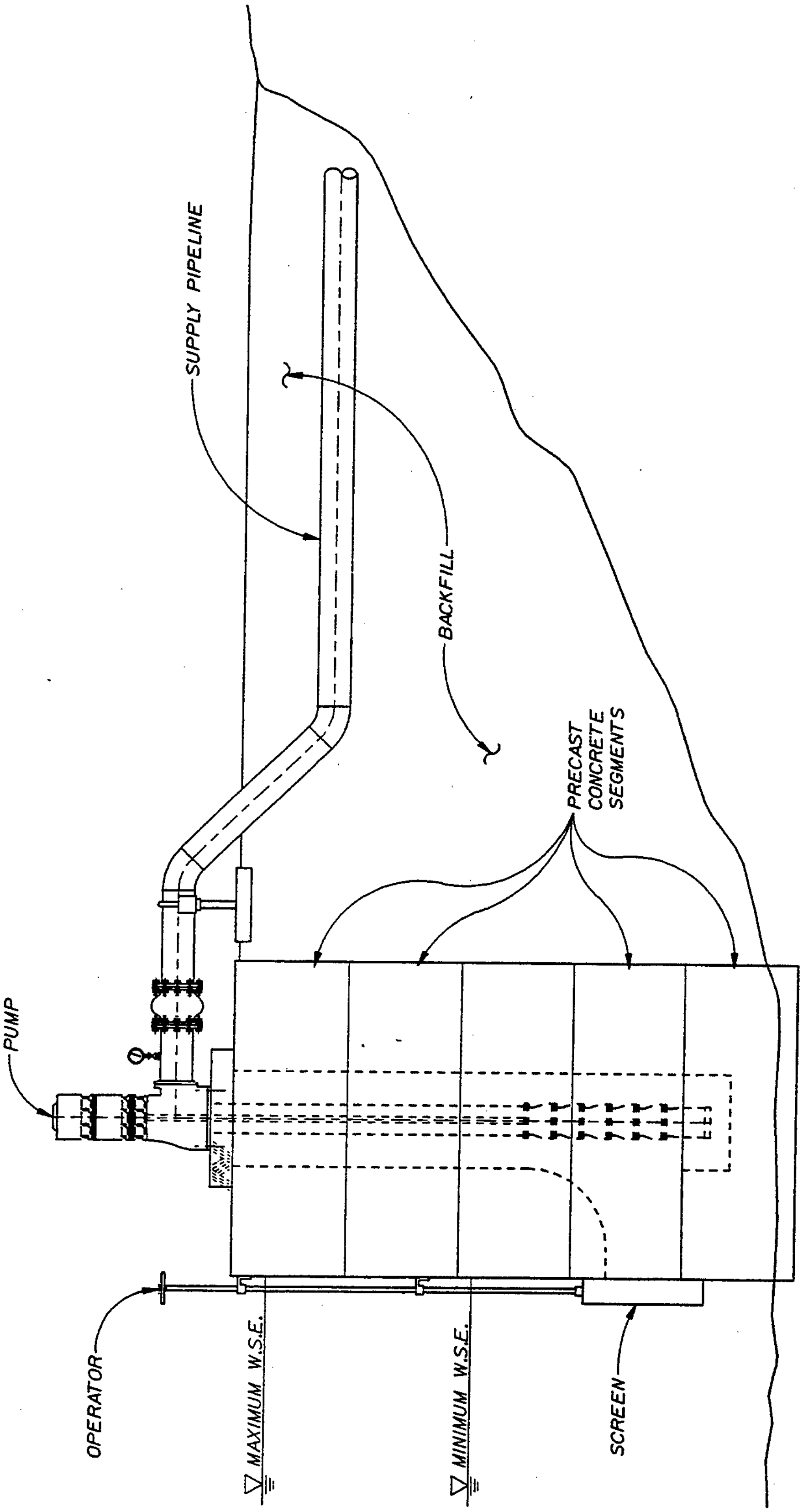
1. Assumed in-line vertical turbine pumps to be used to pump the indicated flow volume from the lowered pool to the existing pumps.
2. Assumed 6 ft/sec velocity in new pipe from new pump to existing pump.
3. Would need electrical modifications also - power supply, cable and controls (may require complete replacement of existing electrical, additional generator, etc.).
4. Would need new intake screens and will need to plumb new pipe from lowered pool into existing pump (may require building/slab modifications).
5. Assumed one new pump for each case shown below.
6. Assumed 1.5 ft head loss per 100 linear feet of pipe.
7. Assumed that the flow data in the table was collected data using the associated measurement. The units were not converted to metric since the data is collected data.

Table J4. HMU Water Well Modifications

Item	Well No. 1	Well No. 3
Well Information		
Location	Ridpath HMU	John Henley HMU
Surface Elevation	198 m	180m
Depth Elevation (BOH)	169 m	145 m
Current River Water Elevation	195 m	165 m
Static Water Elevation	195 m	158 m
Stratigraphy		
Stratigraphy Description	0.0 m to 29.3 m alluvial silt, sand, and gravel underlain by basalt bedrock	0.0 m to 4.6 m silt; 4.6 m to 32.9 m sand and gravel; 32.9 m to 35.0 m basalt bedrock
Bedrock Elevation	167 m	147 m
Pump Test Information		
Pump Size	30 hp turbine	80 hp
Length of Test	48 hrs	4 hrs
Quantity	Up to 800 gpm	600 gpm
Drawdown	0.3 m	0.9 m
Well Development Information		
River Drawdown Elevation	165 m	149 m
RDWE-BOH	4.3 m	4.6 m
Available Water Column	0.0 m	2.4 m
Hole Diameter	0.3 m	0.2 m
Additional Drilling	91.4 m	91.4 m
Quantity Required	340 gpm	450 gpm
Total Head	122 m	126.5 m
Notes	214.5 liters/sec is needed. The current water column is 25.9 m thick and is entirely in alluvial material. Project drawdown would reduce the water column to 4.3 m below the bottom of the hole.	Up to 294 liters/sec is needed. The current water column is 13.7 m with about 11.6 m in alluvium. Drawdown would reduce the amount of water column in alluvium to about 2.4 m.

Table J5. Water Well Pump Modifications

HMU	Flow (gpm)	TDH (ft)	Assumed Pump Efficiency	Calculated HP Required New Pump	Nominal HP Required New Pump
Ridpath	340	561	0.75	64	75
John Henley	450	830	0.75	126	150



PRECAST CONCRETE WATER INTAKE

NOT TO SCALE

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TYPICAL INTAKE SYSTEM

Figure: J1

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Annex K

Reservoir Revegetation Plan

Annex K: Reservoir Revegetation Plan

K.1 General

Following reservoir drawdown, action is necessary to encourage to the initial development of native vegetation and control of soil erosion due to wind and rain. The proposed efforts are to accelerate this development rather than to allow years of slow development and detrimental erosion that would otherwise occur.

The work can be grouped into four distinct phases of work.

- Phase I - Initial seeding to be done during the drawdown period, August to October.
- Phase II - Seeding by drill to be performed the following spring. This seeding is to revegetate areas where seed did not take under Phase I
- Phase III - Manual placement of willow- cottonwood-type plantings. This work would be done the second spring following drawdown.
- Phase IV - Annual efforts to reestablish vegetation in problem or disturbed areas. This work would be initiated during the second spring season and continued on a decreasing rate for a period of 10 years.

Details of these efforts are summarized in Table K1.

K.2 Phase I, Initial Seeding

The reservoir drawdown exposes approximately 8,000 hectares (20,000 acres) of stream bank that needs to be revegetated. The exposure of this land mass occurs over a period of 60 days between August and October. As land is exposed, the water saturation level is high. It dries to an unseedable condition within a few weeks. Seeding of this land mass would be done with four seedings at approximately 2-week intervals during drawdown. Each seeding would consist of aerial application of seed and fertilizer, immediately followed by aerial application of a second seed and fertilizer. Helicopter seeding of the upper land mass exposed during the first 2 weeks of drawdown would take place over the full 225-kilometer (140-mile) reach of the river. The second seeding would commence 2 weeks later, followed on a biweekly basis by the third and fourth seedings.

Seed and fuel staging areas could be setup at each dam site at the helipads or on the landing strips and in the Clarkston area. Approximately 48 kilometers (30 miles) of reservoir could be serviced by these staging areas. Given the range and load capacity of a typical helicopter for this use, intermediate staging areas should be considered.

A special seed blend of native plants would be combined with fertilizer for this seeding. In addition, a seeding of annual cereal grain would be made to allow immediate cover development.

Expectations are low that this seeding will result more than sporatic vegetation. Germination of seed during this timeframe coupled with summer temperatures and moisture conditions will severely limit the success of this effort.

K.3 Phase II, Seeding by Drill

This study team assumed that the aerial seeding would only be effective for 70 percent of the land mass. The remaining 30 percent land mass would require reseeding. This work must be done during the spring growing season. Seeding would be done using a no-til drill since broadcast or til seeding is neither effective nor practical. Approximately 2,428 hectares (6,000 acres) would be seeded in this manner. The team assumed that the field effort for this work would be a 1- month period during March or April.

K.4 Phase III, Willow and Cottonwood Plantings

Observation of the erosion and water surface effects during the first and second spring season would allow identification of locations for willow and cottonwood plantings. Once locations were identified, a program to produce plant cuttings and manually plant them would be undertaken. This work would be done following the second spring freshet during the months of September and October. The study team estimated that 100,000 plantings would be required.

K.5 Phase IV, Revegetation

During the second season, a programmed system of vegetation evaluation would be implemented and periodic revegetation efforts performed. Under ideal conditions, this work effort should decrease as vegetation stabilizes on the river banks and the system becomes self-sustaining. However, extreme events could occur requiring intensive effort to remediate.

Table K1. Summary of Revegetation Quantities

	Phase I (helicopter 2 passes)	Phase II (drill)	Phase III (manual)	Phase IV (drill/manual)
Ice Harbor				
Seeding - acres	4,700	1,410		200
Plantings - each	0	0	25,000	10,000
Lower Monumental				
Seeding - acres	3,800	1,140		200
Plantings - each	0	0	25,000	10,000
Little Goose				
Seeding - acres	6,500	1,950		300
Plantings - each	0	0	25,000	10,000
Lower Granite				
Seeding - acres	4,500	1,350		200
Plantings - each	0	0	25,000	10,000

Annex L

Cattle Watering Facilities Modification Plan

Annex L: Cattle Watering Facilities Modification Plan

L.1 General

Many of the land acquisition agreements for the Snake River reservoirs provide landowners with guaranteed river access for cattle watering. After drawdown, it would not be practical to provide access to the river for cattle watering. Environmental concerns about cattle waste in the river and the need to extend fences out into the river make providing river access impractical. To meet the legal obligation to provide for cattle watering after drawdown, well drilling would be required.

L.2 Methods

This study team evaluated all available water supply alternatives. The basic criterion used in evaluating these alternatives was that the water source provide a dependable water supply for cattle watering. The following approaches were considered:

- Drilling wells at each cattle watering site
- Tying into Habitat Management Unit (HMU) irrigation water supply systems
- Obtaining water from existing wells
- Obtaining water from tributaries to the Snake River

After thoroughly evaluating all of these alternatives, the study team concluded that drilling wells at each watering site is the only alternative that would provide a dependable source of water. The other alternatives were not considered dependable for the following reasons:

- Tributaries to the Snake River may not be dependable water sources during the dry summer and fall months.
- Existing wells would be significantly affected by the drawdown and might become inoperable.
- HMU irrigation systems would operate only during the irrigation season.

L.3 Well Drilling

To provide water for cattle, a well must be drilled and a pump and water tank installed at each of the watering sites. Anticipated well drilling depths range from 21 m (70 feet) to 88 m (290 feet) and are shown in the "well depth" column in Table L1. Subsurface materials would consist of alluvial sand, gravel and silt, and/or basalt bedrock. The study team assumed that most wells would be drilled in the alluvium.

Each site would require an 11,250-liter (3,000-gallon) water tank to provide an adequate volume of water for the cattle. Assuming that the complete recharge time for each tank is 48 hours (1,000 head per day at 7.5 liters [2 gallons] each = 7,500 liters [2,000 gallons] per day), the pump capacity required would be approximately 19 liters (5 gallons) per minute. At this estimated rate of use, the pump would run 40 percent of each day to maintain the desired water volume in the tank. Since most of the cattle watering sites are remote, solar power would be required to operate the pumps. Watering sites that have electricity available are noted in Table L1.

L.4 Schedule

Well drilling at each of the cattle watering sites would be completed prior to drawdown. There is a risk that some of the wells may not provide the necessary volume after drawdown. However, the water volume requirement from each of these wells is very low, thereby reducing that risk.

Table L1. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
				Ice Harbor Reservoir		
104-6a	S1/2 S23, T10N, R32E RM 19					Use location 104-6c.
104-6b	S1/2 S23, T10N, R32E RM 19.1					Use location 104-6c.
104-6c	S1/2 S23, T10N, R32E RM 19.5	Well	2,200	80	250	Recommend combining 104-6a,b, c into one well facility at 104-6c location. Anticipate well mostly in bedrock. Proximite wells encountered water at 250-400 feet. Road access available.
104-8	S.7, T.10N., R.33E., RM 22.2	Well	3,000	80	150	Drill well on landside of railroad. Anticipate drilling mostly in alluvium. Road access available
104-11	S.23, T.11N., R.33E., MP 29	Well	1,400	180	250	Any proposed water line must cross road and railroad. Road access available for drilling. Suggest shallow well. Drilling half in alluvium and half in bedrock anticipated.
104-16	SE1/4S.13, T.11N., R.33E, MP 30	Well	300	10		Recommend using existing well. Construct small overflow line and tank about 300 feet from existing well.
104-20a	NW1/4S.13, T.11N., R.33E., MP 30.8	Well	500	100	170	Recommend drilling shallow well. Drilling anticipated in alluvium. Road access available.
104-10a	S.11, T.11N., R.33E., MP 31.5	Well	900	200	270	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown. Install solar pump and tank
104-10b	SW 1/4S.1, T.11N., R.33E., MP 32.2	Well	1,300	220	290	Recommend drilling a well on gently sloping ground between road and railroad. Well anticipated to drilled mostly in alluvium
104-20b	E1/2S.1, T.11N., R.33E., MP 32.8	Well	2,000	60	130	Recommend drilling a shallow well. Most drilling anticipated in alluvium. Road access provided.
104-13	SW1/4 S.19, T.12N, R34E. MP 35.4	Well	1,000	70	140	Recommend drilling a shallow well. Most drilling anticipated to be in alluvium. Road access provided.
				Lower Monumental Reservoir		
71-6	SE1/4 S34, T12N, R34E	Well	1,400	160	230	Suggest drilling a well to 230 feet. Road access appears to be established. Anticipated subsurface materials to be alluvium.

Table L2 continued. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
104-13a	SE1/4SE1/4 S.27, T.13N., R.43E.					Suggest using 104-13b location and well to serve 13a and b.
104-13b	SE1/4NE1/4 S.26, T.13N., R.34E., MP 43.6	Well				Suggest using 104-13b location and well to serve 13a and b. Establish small line overflow from existing well. The cost of deepening the existing 12" well has been estimated under well development portion of study.
104-13c	N1/2 S.25,T.13N. R.34E. MP 44.3	Well	300	20		Suggest using 104-13c location and well to serve 13b and c. Establish small line overflow from existing well. The cost of deepening the existing 12" well has been estimated under well development portion of study.
104-13d	SE1/4NW1/4NW1/4 S.19, T.13N., R.35E.	Well	1,000	65	135	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown.
104-13e	SE1/4NW1/4 S.19. T.13N., R.35E.	Well	1,000	65	135	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown. Install solar pump and tank
104-15a	NW1/4 S20, T13N, R35E	Well	1,000	65	135	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown. Install solar pump and tank
104-15b	NWSW S21, T13N, R35E		200			Construct small-line overflow from existing pump station to supply water to livestock. Install 200 feet of line
104-15c	NE1/4 S28, T13N, R35E	Line	200			Extent existing irrigation line 200 feet to establish watering facility. Use overflow. Install tank.
104-15d	NE1/4 S.27, T.13N., R.35E.	Well	1,300	100	170	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown. Install solar pump and tank
80-28a	NW1/4NW1/4 S.19, T.13N., R.36E.	Well	700	65	135	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown. Install solar pump and tank
104-10a	NW1/4SE1/4 S.19, T.13N., R.36E	Well	1,000	70	170	Recommend drilling shallow well. All equipment will have to transported by barge and well drilled prior to drawdown.

Table L3 continued. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
104-23	SW1/4 S.16, T13N., R.36E.	Well	1,500	130	200	Recommend drill well to 200 feet and installing solar pump and tank upslope from road.
80-28c	NENE S21, T13N, R36E					Recommend using site 104-10b for watering facility due to proximity.
104 10b	E1/2 S22, T13N, R36E	Well	1,100	80	150	Recommend drilling well to 150 feet. Anticipate alluvium for full well depth. Road access available. Install solar pump and tank.
104-12	NENE S27, T13N, R36E	Well	1,600	200	270	Recommend drilling well to 270 feet. Anticipate alluvium for full well depth. Road access available. Install solar pump and tank.
78-33	NE1/4,SW1/4 S30, T13N, R37E	Well		100	170	Recommend drilling well to 170 feet. Road access available. Anticipate drilling entirely in alluvial material. Install tank and solar pump facility.
104-16a	SE1/4NW1/4 S29, T13N, R37E	Well	700	65	135	Recommend drilling shallow well. No electrical power or road access available. Transport equipment by barge prior to drawdown.
104-3	NE1/4SE1/4, S32, T13N, R37E	Pipe And Tank	300			Area of current reservation is very congested due to highway and railroad. Recommend establishing watering facility near well 257 by constructing small pipe overflow and tank. Use 3,000 gallon tank and 300 feet of piping.
104-16b	NE1/4SW1/4 S33, T13N, R37E	Well	800	40	110	Recommend drilling shallow well. Transport equipment by barge and drill well prior to drawdown. No electrical power or road access available.
104-16c	NE1/4NW1/4 S3, T12N, R37E	Well	600	40	110	Recommend drilling shallow well. Transport equipment by barge and drill well prior to drawdown. No electrical power or road access available.
104-21	SE1/4, NE1/4, S3, T12N, R37E					Very steep access. Reservation seems impractical.
104-10a	SE1/4NW1/4 S11,T12N,R37E	Well	800	20	90	Recommend drilling shallow well and installing pump and tank. Power available adjacent to site. Anticipate alluvial subsurface materials.

Table L4 continued. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
104-9	SE1/4,SW1/4,S31,T13N, R38E	Well	1300	100	170	Recommend drilling well to 170 feet and installing pump and tank. Power available near site. Anticipate alluvium in subsurface.
104-10b	NE1/4,NE1/4,S31,T13N,R38E	Well	1200	100	170	Recommend drilling well to 120 feet and installing pump and tank. Power available near site.
104-4	NW1/4NE1/4 S19, T13N R38E	No Effect				Alkalai Creek
104-18	NE1/4, NE1/4, S30, T13N, R38E	No Effect				Alkalai Creek
104-10c	NW1/4NW1/4 S33, T13N, R38E	Well	1000	130	200	Recommend drilling shallow well. No electrical power or road access available. Transport equipment by barge prior to drawdown.
104-15	NW1/4, NW1/4, S27, T13N, R38E	Well	1200	160	230	Recommend drilling shallow well to 230 feet. Anticipate alluvial subgrade. Power available to the site.
			Little Goose Reservoir			
79-4a	SE1/4NW1/4, S25, T13N, R38E	Well	1500	130	200	Recommend drilling a shallow well to 200 feet and installing a solar pump and tank. Road access available. Alluvial subsurface.
79-4b	NW1/4SW1/4 S30, T13N, R39E	Well	2200	130	200	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
79-4c,d	SE1/4, NE1/4, S30, T13N, R38E					Use 79-4d location to service 79-4c and 4d.
104-3	NW1/4SW1/4 S20, T13N, R39E	Well	800	130	200	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-20	NW1/4SE1/4 S21, T13N, R39E	Well	1500	100	170	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-4a	SW1/4, NE1/4,S.28, T13N. R39E	Well	1000	65	135	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.

Table L5 continued. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
104-4b	NW1/4SW1/4 S27, T13N, R39E	Well	1500	65	135	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-22a	SW1/4NE1/4 S30, T13N, R39E	Well	700	65	135	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-24a	NE1/4SW1/4 S24, T13N, R39E	Pump And Tank	300	10		Install solar pump and tank with N.Y. Gulch Creek as the source of water. Too much distance for line from river after drawdown.
104-24b	CENTER S13, T13N, R39E.	Pipe And Tank	200	10		Recommend using N.Y. Bar well for water supply. Reservation at least 2500 ft from river after dd. Subsurface material is bedrock. Install small-pipe overflow and tank. Power available
104-22b	SW1/4SE1/4 S7, T13N, R40E	Pipe And Tank	200	10		Recommend using existing irrigation lines from New York Bar HMU. Install overflow pipe and tank.
104-24c	SW1/4SW1/4 S8, T13N, R40E	Well	800	65	135	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-22c	NW1/4NE1/4 S16, T13N, R40E	Well	500	40	110	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-7a	NW1/4, NW1/4, S15, T13N, R40E	Well	700	10	70	Recommend drilling a shallow well. Road access available.
104-6	SW1/4, NE1/4, S12 T13N, R39E	Well	1000	100	170	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-7	NW1/4, NW1/4, S15, T13N, R40E	Well	1300	100	170	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-19	N1/2, NE1/4, S 17, T14N, R41E	Pump And Tank	1200	50	120	Recommend installing a pump and tank. Electric power available within 500 feet. No road access.
104-14	SW1/4NW1/4 S25, T14N, R41E	Well	500	80	150	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-12a	NW1/4NW1/4 S29, T14N, R42E	Well	1000	80	150	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-12c	NW1/4NE1/4 S29, T14N, R42E	Well	900	80	150	Recommend drilling shallow well. No road access available. Use locations 104-12a and c and delete location 104-12b

Table L6 continued. Cattle Watering Reservations and Facilities

Facility No.	Location	Facility Type	Horizontal Distance (ft)	Vertical Distance (ft)	Well Depth (ft)	Remarks
				Lower Granite Reservoir		
104-15	SE1/4SE1/4, S33, T14N, R43E	Well	1800	60	130	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
78-16	NW1/4NE1/4 S3, T13N, R43E	Well	1100	80	150	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-3a	UNSURVEYED	Well	800	60	130	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-3b	SE1/4NE1/4 S3, T 12N,R44E	Well	1400	80	150	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-9	SW1/4SE1/4 S2, T12N, R44E	Well	700	60	130	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.
104-16	NW1/4NW1/4 S8, T11N, R45E	Well	1700	90	160	Drill well to level of bottom of main river channel. Access is from Steptoe Creek Road.
104-12a	SE1/4NW1/4 S20, T11N, R45E	Well	1000	90	160	Recommend drilling shallow well and installing pump and tank. Power available nearby.
104-12b	SW1/4SW1/4 S15, T11N, 45E	Well	600	90	160	Recommend drilling shallow well. Transport equipment by barge prior to drawdown.

Annex M

Recreation Access Modification Plan

Annex M: Recreation Access Modification Plan

M.1 General

This study annex concerns the evaluation of the effects of reservoir drawdown on recreation areas and/or boat accesses along the Snake River from its mouth to the Idaho border, approximately 230 river kilometers (140 river miles). Affected reservoirs include Lake Sacajawea (Ice Harbor Reservoir), Lake West (Lower Monumental Reservoir), Lake Bryan (Little Goose Reservoir) and Lower Granite Lake (Lower Granite Reservoir).

M.2 Methods

Each of the 33 recreation areas were evaluated and assigned a ranking of low, medium, or high regarding the potential impact of the drawdown. Table M1 represents modifications to recreation sites. This annex does not attempt to evaluate post drawdown recreation usage at each of the affected recreation areas. That topic is addressed in other sections of this study.

Table M1. Proposed Modifications to Recreation Sites

	Launch Ramp	Parking	Other	Access Road	Parking	Launch Ramp	Other
Lake Sacajawea (Ice Harbor Reservoir)							
North Shore Ramp	2	2800	Toilet, Shelter				
Charbonneau	2	5000	Toilet, Shelter, Swim Beach, Marine Dump Station	1710	5600	2 lane	Toilet
Levy	2	5500	Swim Beach, Handling docks	735	5600	2-lane ramp	Toilet
Fishhook	2		Boat Moorage, Swim Beach and Fixtures		2200	2-lane	
Windust	1		Swim Beach		2200	2	
Matthews	1		Handling docks			2	
Lake West (Lower Monumental Reservoir)							
Devils Bench	2	25-car	2 vault toilets				
Ayers Boat Basin	2		4 handling docks, 2 picnic shelters, vault toilet				
Lyons Ferry State Park		44-car-trailer	Flush toilet facility, Swim beach and fixtures, 4 picnic shelters		8000	2	Vault Toilet
Lyons Ferry Marina	2	44-car-trailer, 22-car	Flush Toilet Facility, Handling docks, Swim beach and fixtures, Restaurant and maintenance building				
Texas Rapids	2			2100		2	
Riparia							
Lake Bryan (Little Goose Reservoir)							
Little Goose Landing	2		Handling docks				
Central Ferry State Park	4	73 car-trailer	Handling docks, Marine dump station	2500	6700	2	Vault Toilet
Garfield County Ramp	2	Gravel	Vault Toilet				
Willow Landing	2		Handling docks, Vault Toilet				

Table M1 continued. Proposed Modifications to Recreation Sites

	Launch Ramp	Parking	Other	Access Road	Parking	Launch Ramp	Other
Illia Landing	2	11-car trailer	Vault toilet, Picnic tables, Mooring docks				
Illia Dunes							
Boyer Park and Marina	3		Flush Toilet, Gas Dock Facility, Marine dump station, Hotel, Restaurant, Handling docks, Swim beach	740	6700	2	Vault toilet
			Lower Granite Lake (Lower Granite Reservoir)				
Offield Landing	1	15 car-trailer, 10 car	Handling dock, Sun shelter, Vault toilet				
Wawawai County Park							
Wawawai Landing	1	27 car-trailer, 28 car	Swim beach and fixtures, Picnic facilities, Chemical toilet	3911	5000	2	Vault toilet
Blyton Landing	1	17 car-trailer, 19 car					
Nisqually John Landing			Picnic facilities, Vault toilet				
Chief Timothy State Park	4	Car-trailer	Handling docks, Marine dump station	1470	6667	2	Vault toilet
Hells Canyon Resort (Redwolf Marina)	2		Marina, Gas dock facility, Marine dump statoin, Log breakwater, Handling docks	611	5000		Vault toilet
Greenbelt Ramp and Boat Basin	2	34 car-trailer, 34 car	Floating boathouse, Marine fueling station, Handling docks	660	5000	2	Vault toilet
Southway Ramp	2		Chemical toilets, Handling docks	1470	5000	2	Vault toilet
Swallows Park			Swim beach facilities, Handling docks	1223	5000	2	Vault toilet
Hells Gate State Park	6	X	Handling docks, Marina, Commercial moorage, Marine fueling station, Marine dump station, Swim beach facilities	1223	6667	2	Vault toilet
Chief Looking Glass Park and Marina	2		marina				
Clearwater Park			X				
Clearwater Ramp						2	

Considering existing site facilities, features, usage, access, and topography, the study team determined the general impacts of drawdown and mitigative actions required to maintain water access to the post drawdown water levels. The team determined that it would be best to close some sites because of the post drawdown water level and/or the site's proximity to dams and other areas that would remain operational. Estimated post drawdown water elevations were derived from 1957 topographic surveys.

M.3 Assumptions

The study team assumed that the Corps would continue to manage and maintain currently owned lands within the study area. It was assumed that lessees would not be responsible for removal of facilities such as marina moorage, restaurants, hotels, or facilities left in-the-dry as a result of drawdown. This annex does not address mitigation to landowners or lessees for direct effects on current businesses and/or personal liabilities.

M.4 Site Modifications

Site modifications include actions such as abandoning part or all of the facility, demolition of part or all of a facility, and relocation boat ramps, parking lots, and visitor facilities. Table M1 summarizes the modifications at each site.

Demolition of facilities is the removal of all buildings, roads, parking lots, utilities, guardrails, light poles and restoring the site to a relatively undeveloped condition. It does not include removal of vegetation. For some recreation areas the entire area is to be demolished. For others, only marinas and boat ramps are to be demolished. Functions such as day use and camping may continue.

For those recreation areas where boat access to the river is necessary, the existing ramp and associated features will be demolished and a new ramp, parking, and facilities constructed as appropriate for the new river conditions.

M.5 Conclusions

Two of the 33 recreation areas evaluated for this study would not be affected by the drawdown; 2 would be closed to river access and 11 would close entirely. The remaining 18 areas would require modifications for river access. Nine marinas or boat moorage facilities would no longer exist. All swimming beaches, as they currently exist, would be impacted due to severe changes in the water elevations. For sites with irrigated lawns, wells or extended river intakes would be required to ensure continued vegetation. Table M1 provides a detailed listing of all the recreation areas evaluated, with the associated site modifications resulting from drawdown.

Annex N

Cultural Resources Protection Plan

Annex N: Cultural Resources Protection Plan

N.1. General

The proposal drawdown lower Snake River reservoirs would require a comprehensive effort to identify cultural resources in the newly exposed reservoir lands, evaluate sites for the eligibility for nomination to NRHP, monitor ongoing effects to sites, and coordinate with consulting parties on needed mitigative and/or protective measures. This would be conducted under a Cultural Resources Management Plan direction designed to address the special issues related to reservoir drawdowns.

Federal agencies have the responsibility to protect and preserve cultural properties. During a drawdown scenario, the Corps would comply with applicable cultural resources laws, such as the *National Historic Preservation Act* (NHPA), the *Native American Graves Protection and Repatriation Act (NAGPRA)*, and the *Archeological Resources Protection Act (ARPA).* The proposed drawdown of the lower Snake River reservoirs would require a comprehensive effort to identify cultural resources in the newly exposed reservoir lands, evaluate sites for their eligibility for nomination to NRHP, monitor ongoing effects to sites and coordinate with consulting parties on needed mitigative and/or protective measures. This would be conducted under a Cultural Resources Management Plan direction designed to address the special issues related to reservoir drawdowns. Because of the potential future knowledge that archaeological sites might be able to provide and the ongoing values they hold, the preference is to protect them in place if feasible. All site protection work undertaken as a result of reservoir drawdown, will be done in compliance with applicable cultural resources laws and regulations. This will include coordination and consultation with the appropriate State Historic Preservation Office and other interested parties such as Tribes and local governments. The Advisory Council on Historic Preservation would also be involved, as appropriate. [See the Cultural Resources Appendix for additional resource management discussions.]

Known cultural resources in the Columbia Basin include archaeological sites as well as traditional cultural properties. Historic settlements by Euro-Americans, Asians, and other non-native peoples are also present. The vast majority of recorded cultural properties are prehistoric sites such as open sites, lithic scatters, rock shelters, pithouses and other depression features, burials, fishing stations, and middens. Prehistoric sites can be classified into five general types - campsites, rock shelters, cemeteries, village sites, and rock art.

Returning the river to near natural levels would completely or partially expose sites that are currently inundated. A total of 360 cultural resources sites are recorded within the four lower Snake River reservoirs. Of this number, 263 are partially or totally inundated and would be directly impacted by drawdown alternatives.

Potential effects on sites exposed by drawdown include vandalism, theft, visual and aesthetic impacts, wind and sheet erosion, animal wallows, animal trampling and burrowing, wet and dry cycles, lateral displacement, wave erosion, slumping, scouring, terracing, and chemical change. Sites would be exposed to these potential impacts year round.

The following discussion is directed towards archaeological sites located on the lower Snake River which at present are either completely or partially inundated. Most of the information on these sites was generated over 25 years ago prior to construction of the dams and reservoirs. It must be understood that this information is both limited in its scope as well as dated. Further, it should also be recognized that

conditions may be far different today than what they were at the time site information was generated. As a result, site protection estimates based on this data may not reflect actual needs.

N.1.1 Approach

Since disclosure of cultural resources site locations is not allowed, this discussion of sites and treatments must be done on a generic basis. For this evaluation, the sites were categorized in three ways. First, the known sites were grouped by reservoir, such as Ice Harbor Reservoir, Lower Monumental Reservoir, and so forth. Next, sites in each reservoir were further categorized by type, such as campsites, villages, rock shelters, and cemeteries. Finally, these subgroups were categorized by the extent of protection they required (i.e., high, medium, low, or no protection). A generic treatment method was formulated for each protection measure. Archaeological site protection measures include data recovery and monitoring. However, for purposes of this discussion, these specific options were considered outside the scope of this study and, therefore, not evaluated.

N.1.2 Definition of Sites

The four types of sites considered for protection measures in this study are campsites, village sites, rock shelters, and cemeteries. Rock art sites would not be protected by measures in this plan and therefore are not included. (**NOTE**: The estimated site areas provided below are based on taking the average of the sum of the total area for each site type for which information was available. However, if there is a drawdown, the actual number and size of sites could change substantially after cultural resources surveys are completed.)

Campsites are areas where people stayed temporarily, without constructing long-term shelter. These sites were usually task oriented and were temporary bases from which to carry out some subsistence task. Examples are hunting camps, gathering camps (camas, berries, couse, etc.), and fishing camps. They are assigned an estimated average area of 100 square meters (m^2) (1,076 square feet [sq ft]).

Villages are long-term habitation sites. They could be permanent or seasonal (often winter) habitation and contained substantial shelters. They were gathering areas for people who may have traveled in smaller groups during part of the year. This provided an average total area for village sites of 300 m^2 (3,229 sq ft).

Rock shelters are natural shelters of various sizes that have been used for food storage, temporary camps, and long-term habitation. Rock shelters were assigned an estimated average area of 20 m^2 (215 sq ft).

Cemeteries are sites where human remains have been intentionally interred or otherwise disposed of. They characteristically hold special cultural significance. Cemeteries are assigned an estimated average area of 100 m^2 (1,076 sq ft).

The percentage of each site type was determined by a count of recorded sites in the four lower Snake River projects. Of the 375 sites potentially affected by the drawdown, the percentages of recorded site types and treatment areas are shown in Table N1. The total site area to be protected is 35,300 m^2 (380,000 sq ft).

Table N1. Classification of Cultural Resource Sites

Site	Percent by Type	Total Sites	Total Area of Sites
Campsites	50%	132	@ 100 m^2 Each = 13,200 m^2
Village Sites	25%	65	@ 300 m^2 Each = 19,500 m^2
Rockshelters/Caves	9%	25	@ 20 m^2 Each = 500 m^2
Cemeteries	8%	21	@ 100 m^2 Each = 2,100 m^2
Rock Art	8% (Not Averaged)	20	
Totals	100%	263	35,300 m^2

N.1.3 Distribution of Sites by Reservoir

There are approximately 375 known archaeological sites located within the four lower Snake run-of-river reservoirs (Lower Granite—136; Little Goose—76; Lower Monumental—103; and Ice Harbor—57). This is a number reflected by the geographic information system database of the Walla Walla District. The number changes periodically when sites are discovered and recorded. Cultural resources will continue to be discovered well into the future as much of the Corps land in the lower Snake River has not been systematically surveyed.

N.1.4 Protection Level Options

The proposed protection options are divided into three ranges that relate to the extent of physical protection. The three ranges generally correlate with expense of implementation. It is difficult to determine the level of protection that would be required by individual cultural sites exposed by a lower Snake River drawdown without an on-the-ground evaluation of the sites. Another problem is that, prior to inundation, no meaningful sample of the inundated sites was evaluated as to eligibility for the National Register of Historic Places. (The National Register is a listing of significant cultural properties from throughout the country.) However, if we assume an eligibility rate for the inundated sites similar to exposed/evaluated sites, workable estimates can be produced. For the purposes of this plan, the study team assumed the following distribution of protection levels:

- 20% of total area to be protected would require a high-range protection.
- 30% would require a medium-range protection
- 30% would require a low-range protection
- 20% would require no physical protection measures because of sediment covering, natural revegetation, inaccessibility, etc.

It should be noted that even with the above distribution of protection levels, site-specific evaluation will have to be completed prior to any site protection work being done as discussed below. Evaluation work (e.g., surveying, mapping, testing) will determine National Register eligibility, site condition, and level of needed/required site protection.

High-Range Protection

This level of protection is intended to protect existing cultural properties against all probable impacts for an indefinite period. This is to be considered a permanent protective measure. Figure N1 illustrates the extent of high-range protection measures.

Example: A site is exposed by drawdown with no vegetation cover and cultural material is exposed on the surface. The site is adjacent to the new water level and will likely be affected by wave action,

scouring, and slumping. The upland portion of the site is exposed to wind and sheet erosion, vandalism, and animal activity.

Possible measures available for high-range protection are as follows:

- Pre-place rock material at or near the site. Possible methods include barge or truck delivery.
- Mobilize loader, materials, personnel, and equipment to site.
- Prepare the site to provide a stable platform for protective structures. Slope cut-banks as necessary to facilitate placement of bank protection material.
- Install a geomembrane filter layer over the entire site.
- Place a shotrock layer over the filter layer.
- Place a riprap or gabion bank protection and groins to protect the slope and prevent back cutting of the armored slope.
- Cover the upland portion of the site with 51 millimeters (mm) (2 inches) of gravel to protect from equipment movement and provide a horizon to indicate the original surface, for sites with truck access.
- Place fill over the upland portion to establish vegetation.
- Revegetate the area to protect against wind and rain erosion.

Medium-Range Protection

There are many measures that can be considered mid-level protection. The archaeological nature of the site, its geography, its geology, and most probable impacts will determine the most appropriate protective measures to use. However, while representing a mid-point in protective effectiveness and cost, mid-level protection measures may not be representative of the actual cost effectiveness of the measure in protecting the cultural property. Figure N2 illustrates the extent of medium-range protection measures.

Example A: A site is exposed by drawdown that is located on nearly level ground. The site has exposed cultural material on the surface and is likely to be affected by wind and rain erosion and vandalism, as well as inadvertent impacts from recreation activities.

Example B: A small site is exposed by drawdown. The site has high significance and cultural sensitivity. The site has received some siltation so there are no cultural properties exposed on the surface. The site is very likely to be affected by vandalism as well as inadvertent damage.

Possible measures to be taken for mid-range protection are as follows:

- Pre-place rock material at or near the site. Possible methods include barge or truck delivery.
- Mobilize loader, materials, personnel, and equipment to site.
- Prepare the site to provide a stable platform for protective structures. Slope cut-banks as necessary to facilitate placement of bank protection material.
- Install a geomembrane filter layer over the entire site.
- Place 305 mm (12 inches) of fill over the upland portion to establish vegetation.
- Revegetate the area to protect against wind and rain erosion.

Low Range Protection Measure

This level of protection is intended to meet the need of temporarily protecting cultural properties from immediate impacts until appropriate permanent protective measures are determined. Figure N3 illustrates the extent of low-range protection measures.

Example: A site is exposed on a level terrace that has received several feet of silt deposit during inundation.

The measures to be taken for low-range protection are as follows:

- Mobilize materials, personnel, and equipment to site.
- Manually grade the site to provide a stable platform for protective structures.
- Establish vegetation over the site.

N.1.5 Implementation Issues

Site Survey

At this time, it is not possible to know the condition of inundated archaeological sites. Furthermore, the process and aftermath of drawdown may reveal the existence of many more sites. While protection activity proceeds for the known sites, it will be necessary to perform a comprehensive survey of each reservoir to identify any additional sites that need protection.

Site Access

The study team assumed that access to the majority of the sites will be possible by land vehicle. A network of county and state highways cross the region. Numerous secondary and unimproved roads provide access to more remote areas. In certain circumstances, some minor overland travel to a site may be possible where no roadway exists. These access points would be minimal and structured so that there are no long-term traces of such access. In some cases the railroad may be a convenient method to attain access to sites. This would require coordination with the appropriate railroad. After the reservoirs are lowered, numerous roadbeds and railroad beds would be available for access to the sites. However, these access ways may not be functional for some months after drawdown.

The study team also assumed for this plan that a few sites will not be accessible by overland vehicle or via the railroad. These sites would be accessed by boat and by helicopter. The protection measures would be modified in these cases to minimize the importation of materials and the use of equipment. It is assumed that equipment would be flown in by helicopter, and materials and personnel would be transported to the site by boat.

Rock Sources

Several rock types are used in constructing these protection measures. Riprap would be used for bank stabilization and groin construction. Smaller shotrock, the waste from riprap production, would be used for the protective layers overlying the geomembrane filters. Highway roadbase material would be used for the interface layers. A very large production and transportation program is included in the work associated with stabilizing the railroad embankments, stabilizing the drainage structures, and constructing the channelization levees. Rock for these operations would be produced at one or more quarry locations with the rock transported by barge, prior to drawdown of the reservoirs, to the specific construction areas along the 225-kilometer (140-mile) river reach. The riprap and shotrock required for cultural resources site protection would be supplied in the same manner during that time. This means that barges would transport and deposit a small quantity of material at the designated location so that equipment can retrieve the material and place it as needed. For sites accessible by road or railroad, centralized material

stockpiles would be made. For the remote sites, site-specific rock deposits would be made. Highway roadbase materials would only be used at sites where vehicle access is possible.

Other Materials and Equipment

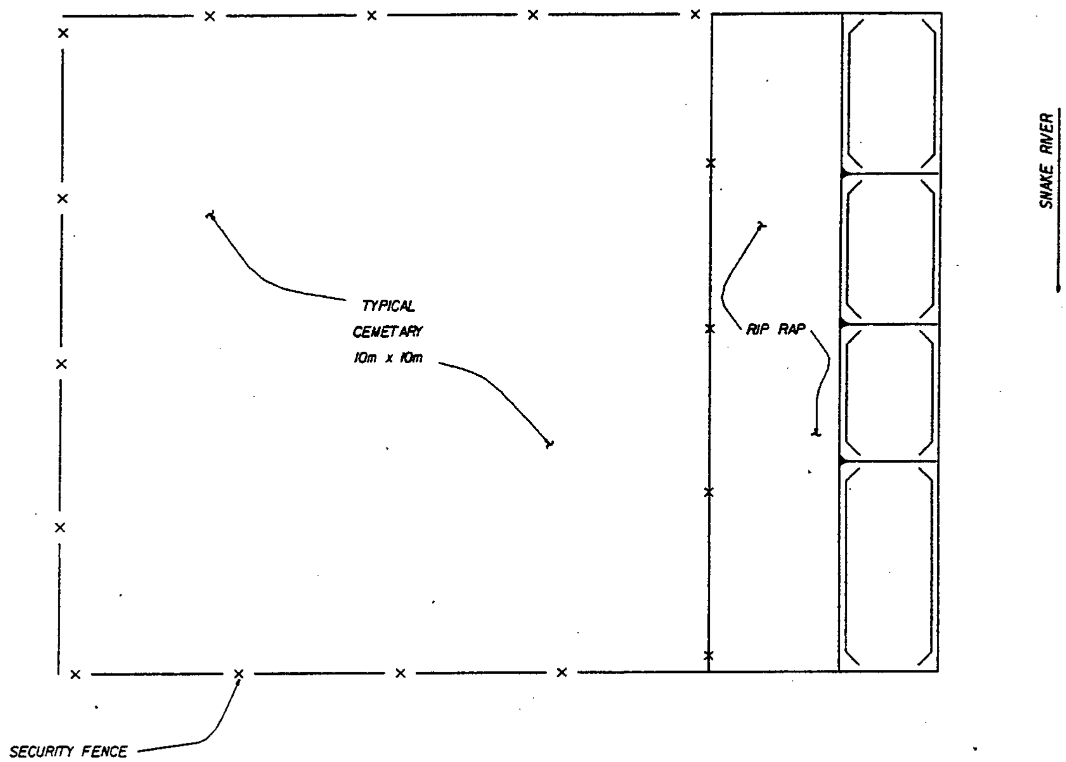
The individual site requirements for other construction materials are relatively minor and do not pose a major logistical effort. The work at each site would require only gross handling of materials. Site grading, excavation, and rock placement would be possible with a small front-end loader similar to a CAT 950. While this piece of equipment would not be most convenient for all operations, this is the best choice if all site work were limited to only one piece of equipment. For remote sites requiring helicopter transportation, the preferred piece of equipment is a small bobcat. This equipment allows lower helicopter rates in exchange for extended production periods on site.

Labor Source

Cost for performing the work was based on utilization of standard contracting processes and regional contractors, regional materials, and prevailing wage and standard equipment rates.

Schedule

Following drawdown and after cultural resources site assessments, work can commence on installing protection measures at identified sites. Although some sites will need immediate work, the time frame for this work could span a period of approximately 10 years. The first year, just prior to the actual drawdown of the reservoirs, would be devoted to developing contract documents to perform the work. Concurrent work would be the pre-placement of rock materials at the various sites under other contracts.



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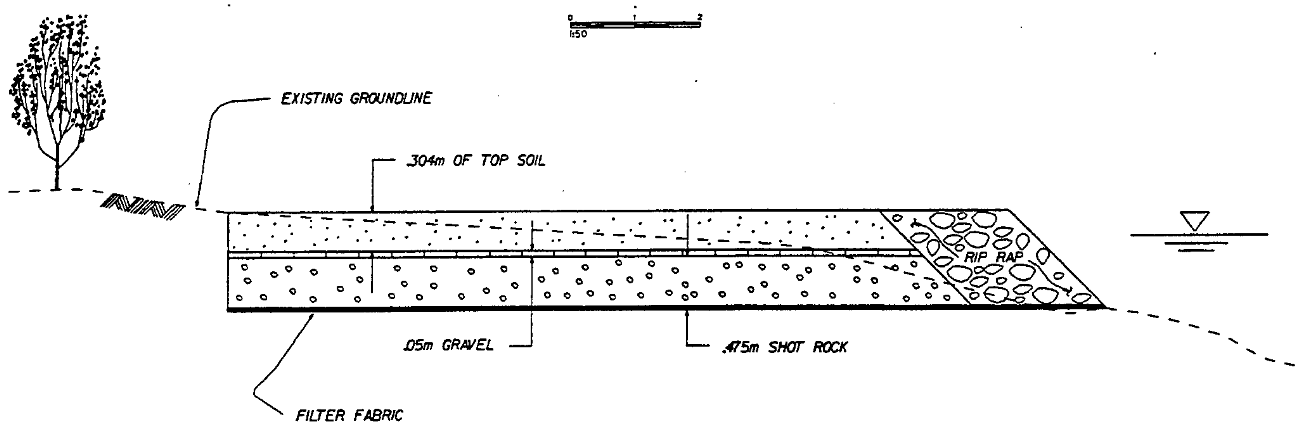

SECTION - CEMETARY

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
CULTURAL RESOURCES PROTECTION PLAN
HIGH RANGE PROTECTION

Figure: N1

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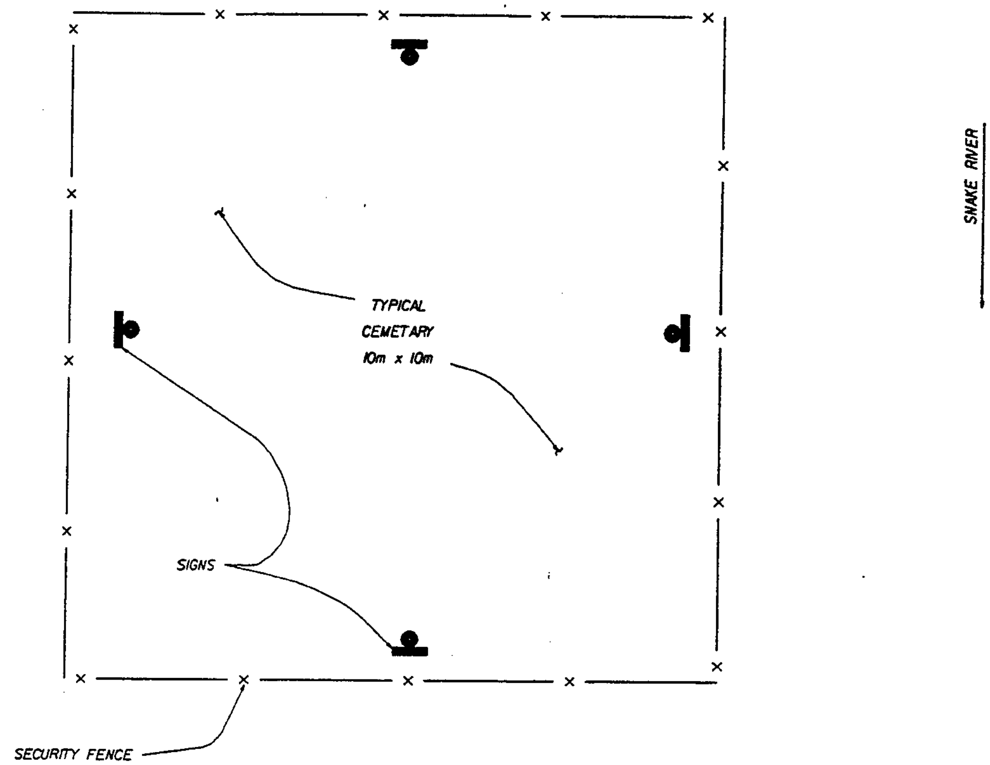

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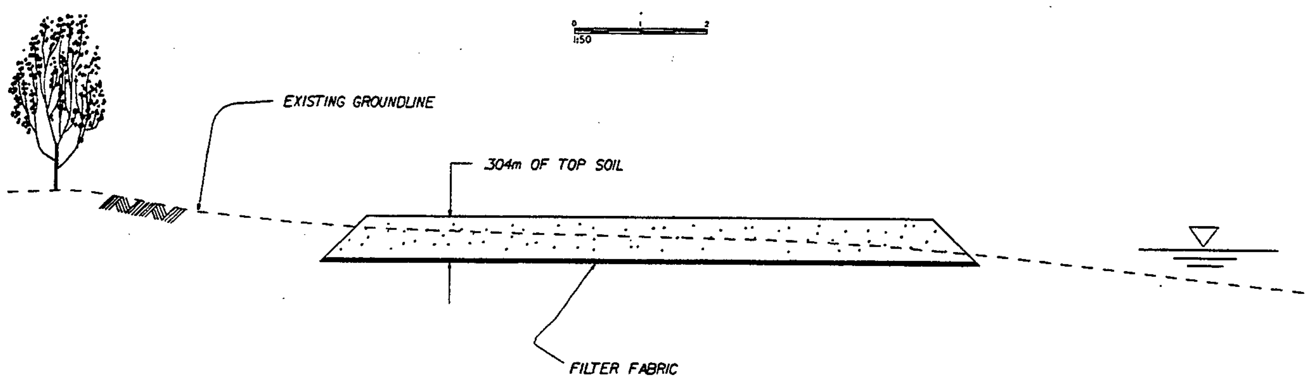

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US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
CULTURAL RESOURCES PROTECTION PLAN
MIDDLE RANGE PROTECTION

Figure:
N2

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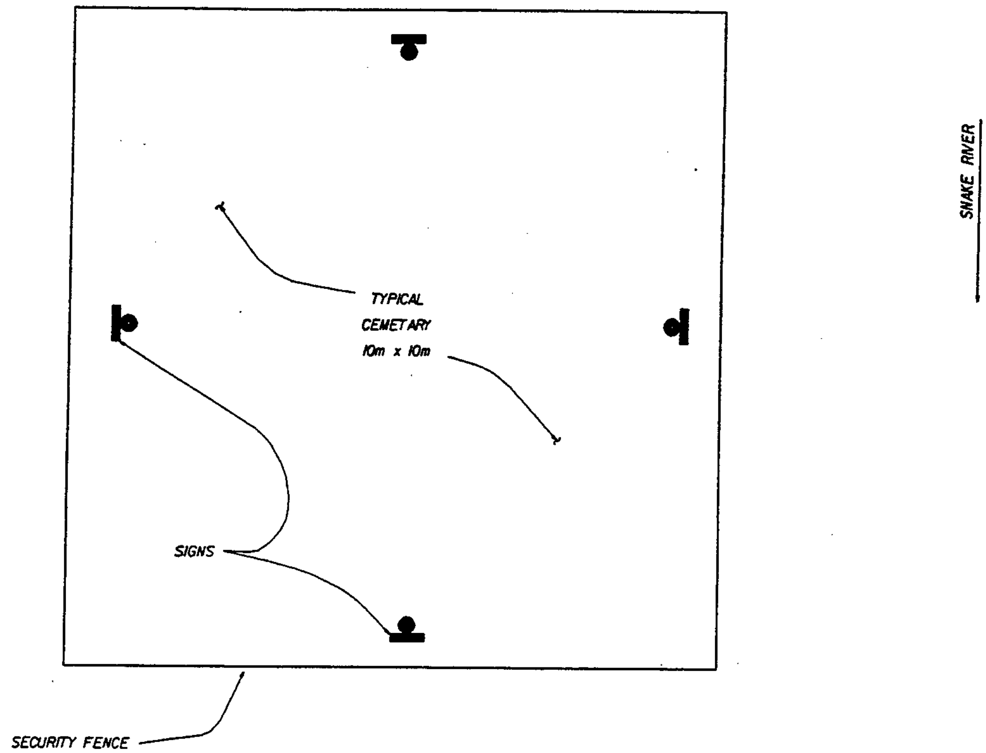

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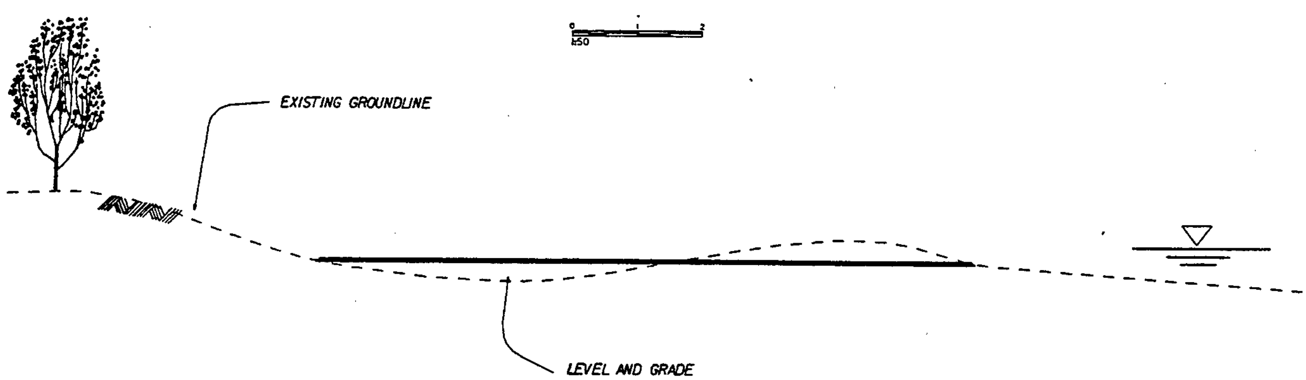

SECTION - CEMETARY

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
CULTURAL RESOURCES PROTECTION PLAN
LOW RANGE PROTECTION

Figure:
N3

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Annex O

Irrigation Systems Modification Plan

Annex O: Irrigation System Modification Plan

O.1 General

This annex addresses modifications needed to the lower Snake River facilities that withdraw water from the Ice Harbor Reservoir for agricultural uses. Modifications described here are not considered as part of the project implementation costs. The plan and costs were developed for economic evaluations of local, regional and national impacts.

Irrigation water facilities for agricultural use are concentrated at the Ice Harbor Reservoir. Of the 19 listed pumping stations and associated operators, 12 pumping stations are currently using Snake River water for agricultural purposes. Table O1 provides a summary of pumping plants, operators, pumping capacity, and irrigated area and crops. Note that several facilities are joint use facilities where two or more operators use one plant site.

The area irrigated by the 12 pumping stations totals approximately 15,000 hectares (37,000 acres) of land. Approximately 11% of the irrigated acreage is used for fruit trees, 6% for grape vineyards, 23% for hybrid poplar and cottonwood harvested for pulp for cardboard manufacture, and 46% for annual row crops. Approximately 14% of the acreage is undefined. A total of 40% of the acreage is used for mature tree-like plants that are not capable of surviving a season without irrigation.

The primary assumption on which this irrigation system modification is based is that the current water demand must be met by a replacement system and be operational prior to the initiation of the drawdown of the Ice Harbor Reservoir. The system must function through a full range of river stages without interruption. The design, operation, or scheduled maintenance must address the presence of large quantities of suspended sediment in the water for extended periods of time for several irrigation seasons.

Table O1. Pump Station Data

Facility No.	Location	Feature	Existing Pump Data			Existing Head	Pumps Peak Req.
			Type	Total Hp	# Pumps		
IH1	NWNE S18, T9N, R32E RM 12	Pump Sta.	Vertical Turbine	2,600	8	360	23,900
IH2	NENE S19, T9N., R32E, RM 11.3	Pump Sta.	Vertical Turbine	4,500	5	260	40,000
IH3	SWSW S36, T10N, R32E, RM 16.9	Pump Sta.	Vertical Turbine	13,500	11	460	85,000
IH4	SESE S8, T9N., R32E RM 13	Pump Sta.	Vertical Tubine	60	2	260	6,000
IH4A	SESE S8, T9N., R32E RM 13	Pump Sta.	Vertical Turbine	1,400	8	250	375
IH5	NENE S19, T9N, R32E RM 12	Pump Sta.	Vertical Turbine	4,700	5	260	36,000
IH6	NWNE S9, T9N, R32E RM 14.4	Pump Sta.	Vertical Turbine	2,260	8	260	13,000

Table O1 continued. Pump Station Data

Facility No.	Location	Feature	Existing Pump Data			Existing Head	Pumps Peak Req.
			Type	Total Hp	# Pumps		
IH7	NENE S19, T9N, R32E RM 12	Pump Sta.	3 Vertical Turbine, 6 Centrifugal	4,900	9	260	35,000
IH8	NESE S23, T10N, R32E. RM 19	Pump Sta.	2 Vertical Turbine		2	260	42
IH9	SESW S24, T10N, R32E RM	Pump Sta.	Vertical Turbine	Same as IH-10			
IH10	SESW S24, T10N, R32E RM	Pump Sta.	Vertical Turbine	4,400	8	410	26,000
IH11	SESW S13, T10N, R32E RM 20.4	Pump Sta.	Vertical Turbine	3,900	6	310	22,500
IH12	NWNE S18, T9N, R32E RM 12	Pump Sta.	Vertical Turbine	included with IH1			
IH13	RM 10.3	Pump Sta.	?	250	2	300	2,500
IH14	SENW S3 T9N, R32E RM 15.3	Pump Sta.	Vertical Turbine	450	2	60	3,800
IH15	NENE S8, T10N, R33E RM 23.6	Pump Sta.	Split -case centrifugal	100	1	60	3,800
IH15	NWSE S4, T10N, R32E RM 24.8	Pump Sta.	Split-case centrifugal	150	1	60	1,500
IH16	NENW S24, T9N, R31E, RM 10.3	Pump Sta.	Vertical Turbine	300	2	360	2,970

O.2 Alternatives

O.2.1 Existing Systems

There are seven privately-owned irrigation pumping stations on the Ice Harbor Reservoir. These pumping stations range in size from a peak pumping capacity of 0.2 cubic meters per second (m^3/s) (5.6 cubic feet per second [cfs]) to a peak capacity of 7 m^3/s (247 cfs). In general, the existing pumping stations draw water through intake screens in the pool and pump the water uphill to corresponding distribution systems. The majority of the pumps are vertical turbine type with a few centrifugal pumps. Without the pool of water created by the Ice Harbor Dam, the intakes to these pumping stations would be completely out of the water and would be unable to lift water from the new, lower water surface.

O.2.2 Discussion of Alternatives

This study team considered several alternative means of providing water to the irrigators. Those alternatives included: 1) relocating the pumping stations to the new shoreline, 2) adding booster pumping stations to pump water from the new shoreline to the existing pumping stations, and 3) building a single large pumping station and distribution system that would serve all of the irrigators.

Alternatives 1 and 2

Alternatives 1 and 2 were not examined in detail for several reasons. After drawdown, the water surface elevation and water depth would vary considerably for unregulated flow conditions in the river. Water surface fluctuations between the mean low water elevation and the 100-year flood range from 3 meters (m) (9 feet) to 5 m (15 feet). Because of these fluctuations in water surface elevation, it would be reasonable to use submersible inline turbine pumps for this application. Passive intake screens, installed on the pump suction piping to prevent both passage of debris and harm to fish would need to be located properly to maintain adequate submergence during low flow conditions. An air-burst back-flush cleaning system would be needed for each submerged screen.

The majority of this stretch of the river has a rather wide, flat bottom with substantial silt, sand, and gravel deposits. It is possible that, as material in the river erodes and deposits, serious problems would occur with this type of pumping arrangement. The river may meander, affecting the availability of water for pumping. Deposited material could reduce intake screen submergence or could cover and plug the screens. Erosion could undermine the pumps, piping, and intake screens, affecting the structural integrity of the system. The submerged equipment would be susceptible to damage due to impact from debris. This type of system, regardless of the sediment concerns, would be difficult to operate and maintain. Finally, in addition to the questionable reliability, installing this type of system prior to or during drawdown would be difficult and costly.

Alternative 3

After considering the alternatives, the study team focused on building one large pumping station and distribution system. The team selected this alternative because it avoided many of the problems associated with the other alternatives. In the vicinity of existing pumping plant IH11, the river is narrow and is contained within steep basalt walls. A review of pre-dam river profiles shows the water to be deep in this stretch of the river during minimal flow conditions. This site lends itself well to installation of a large pumping station. Adequate depth is maintained over the pump bowls even at low flows. The rock channel would minimize erosion, and the higher velocities in this narrow stretch would prevent accumulation of silt, sand, and gravel. The steep walls of the channel enable conventional vertical turbine pumps to be used instead of submersibles.

Providing one large pumping plant to serve all irrigators also has advantages with respect to implementation. The majority of the work on the pumping plant and pipeline could be accomplished prior to drawdown. Connecting the new pipeline system to existing irrigation plants may be accomplished in the off-season prior to drawdown.

Sediment Concerns

It is anticipated that the silt and sand that has accumulated in the reservoirs behind the dams would be eroded and entrained by the faster moving river flows during and after drawdown. It may take several years for this material to be depleted. In addition approximately 3-4 million cubic yards of sediments are added to the system from the Snake and Clearwater Rivers. This poses a significant problem for all water supplies that rely upon the river as a source. Excessive quantities of silt and sand would cause damage to pumps, valves, sprinklers, and other components. Intakes would have to be kept clean and clear. Sand particles are heavy enough that most can be kept out of well-designed pumping systems. The silt, however, may remain suspended for long periods of time, even if pumped into large settling ponds. Removal of suspended particles from the pumped water supply could be accomplished by flocculents, but a chemical treatment plant to treat up to 19 cubic meters per second (m^3/s) (680 cubic feet per second [cfs]) would be impractical to construct and operate. The most practical means of handling sand and silt is to use large settling ponds. Settling ponds would help remove sand passed-on from the pumping plant

at the river as well as some of the suspended silt. No data is available to quantify the expected sediment load in the river. The extent of required settling facilities is pure speculation at this point and will need to be addressed in detail in future design efforts.

O.3 Selected Configuration

O.3.1 General Discussion

The selected primary irrigation system is a pressure supply system that withdraws water from one river location and supplies all the distribution systems. An optional feature is to construct a reservoir for sediment and surge control with a main pumping plant and make appropriate modifications to the river intake plant.

The primary irrigation system consists of five main components: 1) a pumping plant at the river, 2) a piping system, 3) connections to existing irrigation systems, 4) secondary pumping plants, and 5) a control system. The plant at the river lifts to the piping system. At plants IH3 and IH6, the new pipeline crosses the path of the existing irrigation pipelines at an elevation considerably higher than and distant from the existing pumping plants. Instead of extending a branch from the new pipeline down to the existing pumping plants and pumping the water back up to the branch elevation, it makes more sense to abandon the existing pumping plant and construct new secondary plants near the intersection between the new and existing pipelines. A control system would be needed to coordinate pumping activities. Float switches would be needed to start and stop the pumps at the river as the water surface in the settling pond fluctuates. Pressure switches or interlocking relays would be needed to coordinate the main pumps with system demand and the start-up or stopping of pumps at the various irrigation plants.

The optional reservoir requires the addition of four main components: 1) a large settling reservoir, 2) a main pumping plant, 3) reconfigured pumps at the river intake plant, and 4) additional supply piping to the reservoir and additional discharge piping from the reservoir. The plant at the river would lift the river water up to the settling reservoir while the main pumping plant would pump from the settling pond into the piping system.

The pumps in the system must be sized to deliver the quantities of water needed by each irrigator. The size of the motors on the pumps is related to the volume of water being pumped, elevation changes, flow losses in the piping system, and the desired pressure at the ends of the pipe branches. Pipe size can greatly influence flow losses in the system and, thus, the size of the motors needed on the pumps. Large-diameter pipe is desired to reduce motor sizes and power consumption. On the other hand, smaller-diameter piping is desirable to reduce pipe costs for such a long, extensive pipeline. In selecting pipe size and motor size and in determining the number of pumps to use, overall project cost and practicality were considered by thus study team, but only at a cursory level. If the decision is made to drawdown the reservoirs and a pumping system similar to that described below is to be pursued, the cost ramifications should be more carefully reviewed.

O.3.2 Primary Irrigation System

Primary River Pumping Station Description

The intake structure would be divided into five bays or sumps and would have a large horizontal deck upon which the pump motors would be mounted. The peak capacity of the pumping plant is estimated be 7 m^3/s (850 cfs). This peak capacity is 25% greater than the 19 m^3/s (680 cfs) peak irrigation demand in order to provide additional capacity to compensate for pumps that are out of service. Immediately behind the trash racks, bulk-head slots would be placed to allow each bay to be dewatered when needed for

maintenance. Behind the bulkhead slots, vertical traveling debris screens would be installed. The screens would have openings no greater than 2 millimeters (mm) (0.08 inches), as required to control entrance flow velocities for fish. The river-facing surface of the screen would travel upwards, carrying any debris attached to the screen surface to the top of the structure. A water spray system within the screen assembly would clean debris from the screen near the top of the structure. A wetted debris channel within the structure deck would convey debris washed from the screen back to the river.

Vertical turbine pump motors would be secured to the deck above each bay, the pump columns extending down into the sump. The pumps would be divided among the five bays. Each bay would have three 1,500-horsepower (hp) pumps and two 600-hp pumps. Valves would be installed on the discharge of each pump to allow the pump to be isolated from the system for maintenance. The valves would be automatically controlled to open slowly after pump start and close slowly prior to pump shutdown in order to control water hammer and surging. Discharge from each pump would manifold to the connection with the main pipeline. A mobile crane could be used to remove and install pump and system components on an as-needed basis. Sand and silt that accumulates in the pump sumps would be conveyed to one end of the sump and pumped out for disposal. Electrical switchgear would be located at deck level. Pumps would be operated in stages as needed to control pressure and flow. Start-up of the private irrigation pumps that are supplied by this pump station would need to be coordinated to keep demand fluctuations at an acceptable level.

Secondary Pumping Plants

The secondary pumping systems would be comprised of covered slabs with canned vertical turbine pumps. Power supply, switchgear, and control systems would be required. The study team assumed that twelve 1,000-hp pumps would be required for the plant that would replace pump station IH3 and four 400-hp pumps would be required for the plant that would replace pump station IH6. The pump suction piping for both of these plants would be plumbed directly to the new water supply pipe with the discharge plumbed into the existing irrigation piping.

Pipeline Description

In general, the proposed pipeline would follow the south shore of the Snake River. The study team assumed that epoxy-lined and polyethylene-coated steel pipe, conforming to American Water Works Association (AWWA) C200, would be used with 18-m (60-foot) pipe lengths and weld bell ends. At the river pumping station, the discharge piping from the pumps would manifold into the main pipeline. For a 2-meters-per-second (m/s) (6-feet-per-second [ft/s]) target flow velocity in the pipe, 4-m (12-foot) diameter pipe would be needed close to the pumping station. The remainder of the pipeline was sized based upon an 2-m/s (6-ft/s) flow velocity, with pipe size reducing as flow is withdrawn to the various existing pumping plants. Pipe wall thickness was based upon internal pressure and external loading. External loads were calculated assuming the following: 1) the piping would be buried with a cover of 3 feet; HS-20 highway loading might be realized; and pipe bedding and compaction would achieve a soil modulus of at least 7×10^5 Pascal (Pa) (1,000 pounds per square inch [psi].

The pipeline from the river would begin at the pump station near river kilometer 32 (river mile 20), proceeding downstream approximately 1,585 m (5,200 feet) to the branch to IH11. The pipe to IH11 would be 1,067 mm (42 inches) in diameter and would cross the river along the river bottom. The length of this branch was estimated to be 823 m (2,700 feet) to cross to Emma Lake and an additional 1,372 m (4,500 feet) to IH11. The pipe to IH11 would need to be excavated in the river channel and covered with rock.

The pipeline serving the remaining stations would begin at 3,048 mm (120 inches) in diameter. Near IH9, the pipeline pipe would cross a ravine. This crossing may be achieved by suspending the pipeline above

the ravine on piers or excavating and covering the pipe in the ravine bottom. The branch to IH9 is estimated to be 30 m (100 feet) long and would be 914-mm (36-inch) diameter pipe. From the IH9 branch, the main line would reduce to 2,743 mm (108 inches) in diameter and continue along the river bank side of the railroad tracks towards IH3, a length of 3,810 m (12,500 feet). Near river kilometer 28.0 (river mile 17.4) and prior to IH9, the bank of the railroad extends all the way to the river. At this location, complete excavation and burial of the pipe may be impractical. The study team assumed that partial excavation would occur and that cover and rip-rap would be provided along this 305-m (1,000-foot) stretch of the line.

Near river kilometer 27.7 (river mile 17.2), again prior to IH9, the main line would cross beneath the railroad tracks to the south side. The study team assumed that, for this crossing and all subsequent railroad and highway crossings, the main line would pass through a vented casing that extends beyond the limits of the crossing. A 61-m (200-foot) long, 1,829-mm (72-inch) diameter branch would feed water to a secondary pump station for IH3.

After the branch to IH3, the main line would reduce to 2,286 mm (90 inches) in diameter and extend up the mild slope to the top of the bluff above the river over a length of 4,511 m (14,800 feet) to the IH6 branch. The 762-mm (30-inch) diameter, 123-m (400-foot) long branch to the IH6 system would feed a secondary pumping plant.

After the IH6 branch, the 2,469-m (8,100-foot) main line would remain 2,286 mm (90 inches) in diameter and would continue along the bluff, angling down toward the river and the IH4 plant. The 30-m (100-foot) long, 762-mm (30-inch) diameter branch to IH4 need not cross under the tracks because the main portion of the IH4 plant is south of the tracks.

After branching to IH4, the main line would reduce to 2,134-m (84-inch) diameter pipe and would continue 1,067 m (3,500 feet) to the IH1/IH12 branch. Shortly after the IH4 branch and prior to the IH1/IH12 branch, the main line would again cross under the railroad tracks. The 1,372-m (4,500-foot) long, 914-mm (36-inch) diameter IH1/IH12 branch would extend across the river bottom and discharge into the north shore bay in which the IH1/IH12 plant is installed. The study team assumed that this bay would be sealed to act as a reservoir for the IH1/IH12 plant, although it may prove to be more cost effective to extend the piping all the way to the pumping plant and manifold the supply directly to the pumps.

After the IH1/IH12 branch, the main line would reduce to 1,981 mm (78 inches) in diameter and continue 701 m (2,300 feet) to its termination at the IH2/IH5/IH7 plant. The majority of the IH2/IH5/IH7 branch would parallel existing gravel and paved roadways, having to cross a paved roadway at one location. Buried utilities would likely be encountered along the route of the IH2/IH5/IH7 branch.

Pipeline Specials

Two of the existing pumping plants, IH6 and IH4, are multi-pump configurations. These plants use small pumps at the river to lift water to main pumping plants that are a short distance further up the shore. The piping from the small plant is plumbed directly into the suction piping of the main plant. This type of arrangement is proposed for connecting the new water supply to each of the affected pumping plants, plants IH6 and IH3 excluded. For plant IH4, the small plant at the river would be abandoned and the existing manifold at the main plant would be connected to the new branch pipe. For plants IH11, IH9, IH1/IH12, and IH2/IH5/IH7, manifolds need to be constructed and installed to connect each pump to the branch piping. The study team assumed that the manifolds would be simple horizontal pipes with vertical branches extending up to and connecting with the bottom of each pump. Some structural modifications

would be needed for the plants that need manifolds. Typical structural modifications would include boring through concrete walls in the existing sumps and providing supports for the manifolds.

In the branch piping near each existing plant and each secondary, an isolation valve would be required to allow the plant to be isolated from the supply system as needed for plant maintenance. The valves would need to be the slow opening and closing type to prevent surging. The study team assumed that manual valve operators would be provided. The team also assumed that a flow meter would be needed in each pipe branch in order to monitor water consumption.

At each branch pipe and at each significant change in direction, the piping would need to be constrained against thrust. The team assumed that concrete thrust blocks would be used to accomplish this. At the new pumping plant near river kilometer 32 (river mile 20), the main line would need to be constrained along the entire length of the intake structure due to the thrust generated by flow from the pump manifolds.

At all high points, large air release/vacuum valves (ARVs) would be required. The ARVs need to be sized to suit the pipe and flow. It is anticipated that at least six locations would require ARVs.

At all low points, drain valves and drain discharge piping would be required to allow the pipeline to be drained. It is anticipated that at least six 610-mm (24-inch) diameter drain valves and piping and at least four 305-mm (12-inch) diameter drain valves and piping would be required.

O.3.3 Optional Reservoir

The sediment concentration in the river is difficult to determine. The ability of the irrigation system to handle high volumes of sediment in the supply water is questionable. Ideally, a holding pond sized to detain the water for a sufficient time to allow settling of suspended solids is desired. A reservoir to provide a significant detention time for peak flows of 19 m^3/s (680 cfs) would be sizeable. The study team estimated that a reservoir of approximately 396 m^3 (14,000 acre-feet) would be required. This assumes an active storage volume of 50 percent and a detention time of 5 days. More detailed evaluation of sediment characteristics may indicate that more advanced water treatment is necessary to remove sediments from the water.

This study team assumed that the area just to the east of the river intake would be the site of the proposed reservoir. The reservoir would be excavated and enclosed using earthen dikes. The reservoir area would be lined with a geomembrane liner to prevent excessive seepage of the stored water. The liner would be subsequently covered with a protective layer of fine-grained material. More detailed evaluations and reconfiguring of the intake and irrigation system may allow advantageous use of existing topographic features to better site a reservoir.

In order to incorporate an in-line reservoir, the river intake would be reconfigured so that each bay would have three 2,500-hp pumps. This horsepower is based upon the settling reservoir having a mean water surface elevation of 213.4 m (700 feet). At the river pumping station, the discharge piping from the pumps would manifold into the main pipeline that feeds the settling reservoir. The pipeline from the river begins at the pump station near river kilometer 32 (river mile 20), angling up the hill approximately 671 m (2,200 feet) and discharging into the settling reservoir above.

To provide a range of flows between partial and full irrigation demand, numerous pumps at the main pumping station would be required. The study team selected an arrangement of 15 250-hp pumps and 10 150-hp pumps. For 18 m (60 feet) of head, the 250-hp pumps would each provide 51 m^3/m (13,440 gpm), and the 150-hp pumps would each provide 25 m^3/m (6,720 gpm). The head required of these pumps is

small because the settling pond elevation is equivalent to the highest anticipated elevation of the pipeline. The 18 m (60 feet) of design head assures at least 6 m (20 feet) of surplus head that exist along the entire length of the pipeline. The study team assumed that canned vertical turbine pumps would be used in order to take advantage of the greater efficiencies possible with this type of pump. The pumps would be supported from a slab with water reaching the pump intakes through buried piping that extends out into the settling pond.

It should also be noted that the use of variable frequency driven pumps to reduce the number of pumps needed to cover a broad range of demand is another alternative that could be very advantageous and should be examined in more detail if the drawdown proceeds.

Debris would inevitably be encountered in the settling reservoir. If the debris were sucked into the pumps, damage could occur. Therefore, the study team assumed that intake screens would be required. Manual cleaning of the intake screens should suffice since debris loads should not be heavy. Power supply, switchgear, and control systems would be required. A roof with removable sections for pump access should be provided to shelter the pumps from the elements and allow pump maintenance.

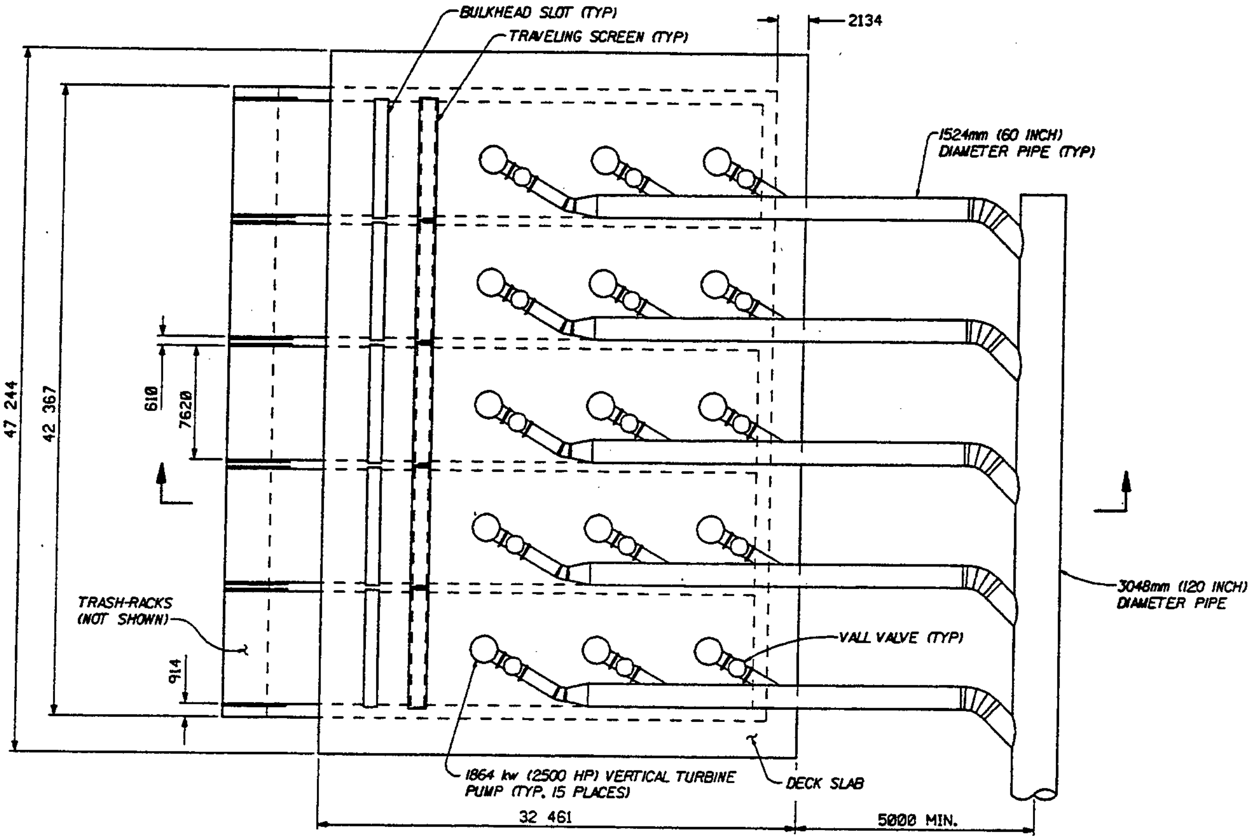
A single pipeline would discharge water from the settling pond to the branch near plant IH11. The 1,067-mm (42-inch) line would then branch to IH11 across the river, and the mainline would continue on to serve the remaining plants

O.3.4 Maintenance Requirements

The extent of increased maintenance activity to treat sediment related problems is not known. Certainly replacement of the wear parts of the pumps, valves, sprinklers, and filters would initially be at a high frequency. Even in later years a higher frequency of parts replacement could be expected with the river in its natural state.

O.4 Schedule

Construction activities for this system must be completed by January of the year in which drawdown occurs. Each irrigator must start the irrigation season on the new system. Drawdown begins in early August of that year, the period of peak water demand. In order of accomplish this, construction of the river intake, the pipeline, and the optional reservoir must commence 24-36 months in advance of the January completion date.



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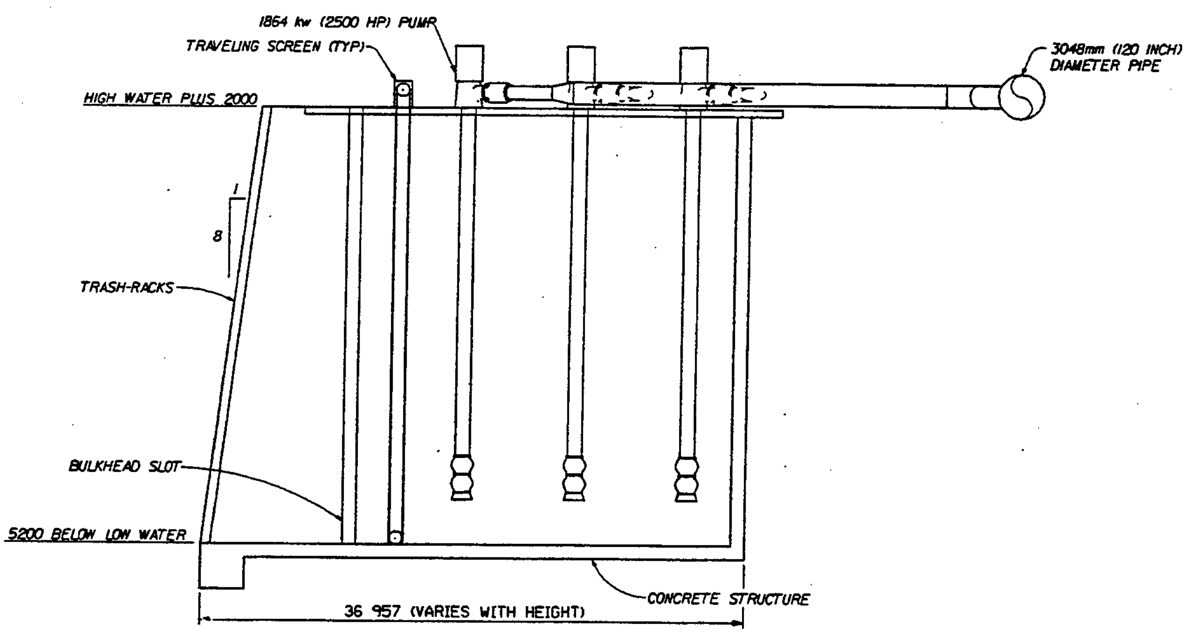

SECTION
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US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
24 CMS (850 CFS) PUMPING PLANT
IRRIGATION SUPPLY

Figure:
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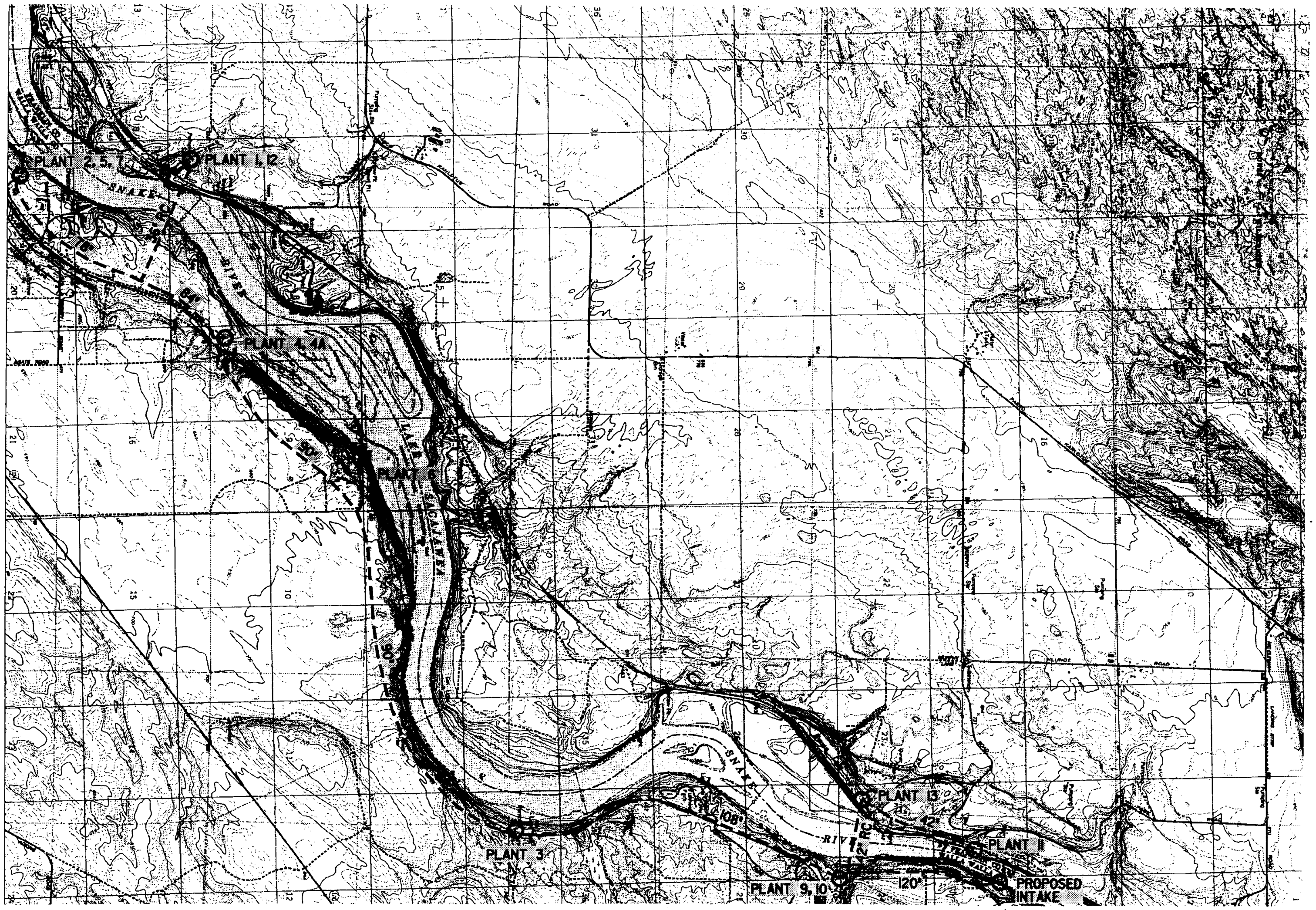

US Army Corps of Engineers
Walla Walla District

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

PIPELINE ROUTING MAP

GRID NORTH
LAMBERT WASH. SOUTH

Figure: 02

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Annex P

Water Well Modification Plan

Annex P: Water Well Modification Plan

P.1 General

Water wells existing along the lower Snake River supply domestic water, agricultural water, and some commercial uses. This annex addresses modifications required for the water wells that may be affected by reservoir lowering. (Annex O discusses modifications required for the agricultural water supply for irrigation users on the Ice Harbor Reservoir.) Modifications to water wells are not considered as part of the project implementation costs. The plan and costs were developed for economic evaluations of local, regional and national impacts.

The water wells evaluated for this study range from shallow wells collecting water from surface sources to deep wells drawing from the deep basalt formations. Drawdown of the water surface in the four lower Snake River reservoirs ranges would result in a water surface change of only less than a meter (a few feet) at the upper reservoir locations to as much as 30 m (100 feet) upstream of each dam site. The aquifers adjacent to the river could be greatly affected by the change in water surface. The degree of impact would depend, in part, on the geologic formation supplying the water to the well, the proximity of the well to the river, and the depth of the well. While it is not possible to characterize each well along the affected river reach, the study team believes that the most adverse effect from drawdown would be to wells drawing water from the shallow aquifers.

Water users whose wells are affected by reservoir drawdown have few options for water during drawdown. Drilling other wells in advance of drawdown is not a viable option since groundwater conditions cannot be accurately predicted. It is highly probable that groundwater water users will experience of loss of water after drawdown and may never be able to restore groundwater.

This report summarizes the method by which the study team determined an estimate of well modifications and presents plans for a reasonable modification to those wells. The study team also determined cost estimates for the modifications needed to maintain the current water supplies.

P.2 Methods

To begin this effort, the study team developed an inventory of the existing water wells within approximately 1.6 kilometer (1 mile) of the Snake River based on information presented on the logs of the drilled wells as recorded by the Washington Department of Ecology, Spokane Office. Approximately 180 water wells are recorded in the designated study area. Since it was not feasible for this study team to perform a detailed evaluation of each well, the team analyzed a representative sampling of the 180 recorded wells.

Of the approximately 180 wells distributed over the area, a representative sample of 38 wells was selected and analyzed. The water well locations, as presented on the well logs, were plotted on U.S. Geological Survey topographic maps, 7.5 minute, 1:24,000 scale. The well log data coupled with topographic features of the area provided information on well depth, stratigraphy, surface elevation, and, ultimately, which wells would likely be affected by the change in water surface elevation. It should be noted, however, that the response of the aquifers to variations in water surface is a complex relationship, and detailed analysis of that relationship was far beyond the scope of this task.

The study team determined that only 15 of the 38 wells in the representative sample would potentially be affected by the drawdown of the four lower Snake River dams. The resulting information, shown in Table P1, is presented in tabular form for convenience and for ease of reference.

For each of the affected wells, the study team determined that modifications should include increasing the depth of the well below the estimated new groundwater surface and installing a new pump and associated hardware to pump against the increased head. This additional depth of drilling and pumping head data is also shown in Table P1. The required pump size, calculated from the information in Table P1, is shown in Table P2 for the 15 modified wells.

P.3 Conclusion and Recommendation

Based on the representative sample results, 71 of the 180 wells (39 percent) would need modification as a result of lower reservoir water surfaces. It is very difficult to determine how much each well would produce after drawdown, or how deep they would need to be drilled to produce water at pre-drawdown rates. Therefore, this study team recommends that all well modifications be performed after drawdown has occurred.

The cost estimate does not include provisions or costs for providing temporary water to users during the period after drawdown and when well can be re-established. The potential water usage during that time period is uncertain.

P.4 Construction Schedule

All well modifications would be performed after drawdown. The installation and subsequent performance of new wells cannot be determined until the postdrawdown groundwater conditions have stabilized.

Table P1. Summary of Data Concerning Wells Sampled

Well Information							Pump Test				New Well Development								Remarks
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head		
Ice Harbor Reservoir																			
97	T.9N.,R.31E.	490	348	440	410	alluvium to 360 ft. el.	100 hp turbine	4 hrs.	1,200 gpm	21 ft.	330	-18	12 in.	0	250	1200 gpm	390 ft.		Drawdown is anticipated to lower the water table 18 feet below the current BOH. Additional drilling would be needed.
95	T.9N.,R.31E.	510	368	440	430	alluvium to 380 ft. el.	100 hp turbine	4 hrs.	1,000 gpm	15 ft.	330	-38	12 in.	0	250+	1000 gpm	390 ft.		Drawdown is anticipated to lower the water table 38 feet below the current BOH. Additional drilling would be needed.
99	T.9N.,R.31E.	470	370	440	427	alluvium to 375 ft. el.	350 hp.	2 hrs.	3,000 gpm	14 ft.	340	-30	16 in.	0	250	3000 gpm	350 ft.		Drawdown is anticipated to lower the water table 40 feet below the BOH. No water column would remain and additional drilling would be needed.
195	T.9N., R.32E.	570	320	440	405	overburden to 473 ft. el.	75 hp	Not Avail -able (NA)	550 gpm	NA	350	30		30	200	550 gpm	400 ft.		Drawdown is anticipated to lower the water table to an elevation 30 feet above the BOH. An additional 100 feet of drilling would be needed.
206	T.9N., R.32E.	630	230	440	480	overburden to 595 ft. el	NA	1.5 hr,	12 gpm	360 ft	350	120		120	0	12 gpm			Drawdown is expected to lower the water table to an elevation 120 above the BOH. No additional drilling anticipated.
198	T.9N., R.32E.	650	323	440	540	overburden to 598 ft. el.	NA	NA	60 gpm	NA	350	27		27	0	60 gpm			Drawdown is anticipated to lower the water table to an elevation 27 feet above the BOH. No additional drilling would be needed.
187	T.9N., R.32E.	510	255	440	368	overburden to 492 ft. el.	airtest	1 hr.	20 gpm	NA	350	95		95	0	20 gpm			Drawdown is expected to lower the water table to an elevation 146 feet above the BOH. No additional drilling is needed.
184	T.9N., R.32E.	480	270	440	390	overburden to 358 ft. el.	pump	NA	700 gpm	40 ft.	350	80		80	0	700 gpm			Drawdown is expected to lower the water table to an elevation 80 feet above the BOH. No additional drilling is needed.

Table P1 continued. Summary of Data Concerning Wells Sampled

Well Information							Pump Test				New Well Development							Remarks
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head	
182	T.9N., R.32E.	460	334	440	410	alluvium to 334 ft. el.	none	none	none	none	350	16	16 in.	16	150	1,080 gpm	276 ft.	Drawdown is expected to lower the water table to an elevation 16 feet above the BOH. 50 feet of additional drilling is needed to accommodate seasonal river level fluctuations.
208	T.10N., R.32E.	760	337	440	NA	overburden to 756 ft. el.	airtest	4 hrs.	30 gpm	NA	350	13	8 in.	13	100	30 gpm	500 ft.	Drawdown is expected to lower the water table to an elevation 13 feet above the BOH. Static water level is not dependent on river water level. No additional drilling anticipated.
215	T.10N., R.32E.	480	360	440	417	alluvium to BOH	NA	48 hrs.	500 gpm	1 ft.	350	-10	16 in.	0	50	300 gpm	170 ft.	Drawdown is expected to lower the water table to an elevation 10 feet below the BOH. It is anticipated that at least 50 feet of additional drilling is needed to maintain water supply.
218	T.10N., R.33E.	480	260	440	440	alluvium to BOH	Layne Pump	8 hrs.	4,400 gpm	25 ft.	380	120		120	0	4,400 gpm		Drawdown is expected to lower the water table to an elevation 120 feet above the BOH. It is anticipated that no additional drilling is needed to maintain water supply.
220	T.10N., R.33E.	920	680	440	880	overburden to 850 ft. el.	NA	NA	NA	NA	380	-300		0	0	100 gpm		Drawdown is expected to lower the water table to an elevation 300 feet below the BOH. It is anticipated that no additional drilling would be required to maintain water supply. The current BOH is 240 above pool and is not dependent on river level.
228	T.11N., R.33E.	450	224	440	NA	alluvium to 395 ft. el.	bailed	NA	17 gpm	0 ft.	390	166		166	0	17 gpm		Drawdown is expected to lower the water table to an elevation 166 above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.

Table P1 continued. Summary of Data Concerning Wells Sampled

	Well Information						Pump Test				New Well Development							Remarks
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head	
231	T.12N., R.34E.	540	338	440	436	overburden to 426 ft. el.	3 hp.	8 hrs.	43 gpm	0.2 ft.	440	102		102	0	43 gpm		Drawdown is expected to lower the water table to an elevation 62 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
Lower Monumental Reservoir																		
235	T.12N., R.34E.	470	370	440	434	alluvium to 371 ft. el.	127 hp.	70 hrs.	300 gpm	0 ft.	420	50		50	0	300 gpm		Drawdown is expected to lower the water table to an elevation 50 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
239	T.13N., R.34E.	590	476	540	515	alluvium to 476 ft. el.	165 hp	NA	1500 gpm	10 ft.	445	-31	12 in.	0	250	1000 gpm	365 ft.	Drawdown is expected to lower the water table to an elevation 81 feet below the BOH. It is anticipated that at least 250 feet of drilling would be required to maintain water supply.
251	T.13N., R.35E.	525	235	540	490	overburden to 380 ft. el.	15 hp.	NA	130 gpm	34 ft.	445	240		240	0	130 gpm		Drawdown is expected to lower the water table to an elevation 240 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
255	T.13N., R.36E.	595	420	540	467	alluvium to 442 ft. el.	15 hp.	22 hrs.	215 gpm	4 ft.	445	25	8 in.	25	200 ft .+	170 gpm	375 ft.	Drawdown is expected to lower the water table to an elevation 25 feet above the BOH. It is anticipated that at least 200 feet of drilling would be required to maintain water supply.
264	T.13N., R.37E.	640	318	540	573	alluvium to 283 ft. el.	NA	40 hrs.	5700 gpm	3.5 ft.	490	172		172	0	5700 gpm		Drawdown is expected to lower the water table to an elevation 209 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.

Table P1 continued. Summary of Data Concerning Wells Sampled

		Well Information					Pump Test				New Well Development							
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head	Remarks
273	T.13N., R.37E.	590	452	540	516	alluvium to 452 ft. el.	NA	8 hrs.	750 gpm	0 ft.	490	40		40	0	750 gpm		Drawdown is expected to lower the water table to an elevation 40 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
279	T.13N., R.38E.	590	475	540	520	alluvium to 482 ft. el.	80 hp	4 hrs.	600 gpm	3 ft.	490	15	8 in.	15	150	450 gpm	265 ft.	Drawdown is expected to lower the water table to an elevation 15 feet above the BOH. It is anticipated that 150 feet of additional drilling is required to maintain water supply.
282	T.13N., R.38E.	590	395	540	500	overburden to 61 ft. el.	3 hp	2 hrs.	49 gpm	10 ft.	490	95		95	0	50 gpm		Drawdown is expected to lower the water table to an elevation 95 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
Little Goose Reservoir																		
286	T.13N., R.39E.	690	370	640	590	overburden to 131 ft. el.	NA	NA	1,036 gpm	23 ft.	540	170		170	0	1,036 gpm		Drawdown is expected to lower the water table to an elevation 170 feet above the BOH. It is anticipated that no additional drilling would be required to maintain water supply.
294	T.13N., R.40E.	765	550	640	615	overburden to 764 ft. el.	bailer	4 hrs.	15 gpm	0 ft.	540	-10	8 in.	0	100	4 gpm	365 ft.	Drawdown is expected to lower the water table to an elevation 10 feet below the BOH. It is anticipated that at least 100 feet of additional drilling is required to maintain water supply.
300	T.13N., R.40E.	655	590	640	618	overburden to 600 ft. el.	1 1/2 hp	4 hrs.	30 gpm	1/2 ft.	540	-50	8 in.	0	150	300 gpm	215 ft.	Drawdown is expected to lower the water table to an elevation 50 feet below the BOH. It is anticipated that at least 150 feet of additional drilling is required to maintain water supply.

Table P1 continued. Summary of Data Concerning Wells Sampled

		Well Information					Pump Test				New Well Development								
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head	Remarks
290	T.13N., R.40E.	655	430	640	515	overburden to 488 ft. el.	NA	NA	600 gpm	11 ft.	540	110		110	0	600 gpm		Drawdown is expected to lower the water table to an elevation 110 feet above the BOH. It is anticipated that no additional drilling is required to maintain current water supply.
317	T.14N., R.40E.	650	-76	640	600	overburden to 550 ft. el.	150 hp	4 hrs.	2090 gpm	80 ft.	540	615		615	0	2,090 gpm		Drawdown is expected to lower the water table to an elevation 615 feet above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
315	T.14N., R.40E.	660	460	640	575	overburden to 535 ft. el.	airtest	1 hr.	75 gpm	NA	540	80		80	0	100 gpm		Drawdown is expected to lower the water table to an elevation 80 above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
322	T.14N., R42E.	645	555	640	615	overburden to 555 ft. el.	NA	NA	37 gpm	13 ft.	540	-15	6 in.	-15	150	150 gpm	240 ft.	Drawdown is expected to lower the water table to an elevation 15 feet below the BOH. It is anticipated that 100 feet additional drilling is needed to maintain current water supply.
328	T.14N., R.43E.	670	495	640	635	overburden to 605 ft. el.	NA	32 hrs.	300 gpm	40 ft.	540	45		45	0	300 gpm		Drawdown is expected to lower the water table to an elevation 45 feet above BOH. It is anticipated that no feet additional drilling is needed to maintain current water supply.
Lower Granite Reservoir																		
324	T.13N., R.43E.	840	590	740	780	overburden to 825 ft. el.	bailer	1 hr.	20 gpm	60 ft.	630	40	10 in.	40	100	34 gpm	350 ft.	Drawdown is expected to lower the water table to an elevation 40 above the BOH. It is anticipated that 100 feet of additional drilling is needed to maintain current water supply. Post-drawdown river level is 70 feet below current water zone in well.

Table P1 continued. Summary of Data Concerning Wells Sampled

Well Information							Pump Test				New Well Development							
Well Number	Location	Surface Elevation	Bottom of Hole (BOH)	Current River Water Elevation	Static Water Level (SWL) Elevation	Stratigraphy	Pump Size	Length Of Test	Quantity	Drawdown	Drawdown Water Surface Elevation (DWSE)	DWSE-BOH	Hole Diameter	Avail. Water Column	Additional Drilling	Quantity Required	Total Head	Remarks
340	T.11N., R.45E.	750	370	740	490	overburden to 748 ft. el.	airtest	1 hr.	6 gpm	NA	680	310		310	0	6 gpm		Drawdown is expected to lower the water table to an elevation 310 feet above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
343	T.11N., R.45E.	750	450	740	590	overburden to 730 ft. el.	airtest	1 hr.	30 gpm	NA	700	250		250	0	30 gpm		Drawdown is expected to lower the water table to an elevation 250 feet above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
348	T.11N., R.45E.	750	635	740	720	overburden to 655 ft. el.	NA	NA	NA	NA	700	65		65	0	100 gpm		Drawdown is expected to lower the water table to an elevation 65 feet above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
356	T.11N., R.45E.	1150	870	740	965	overburden to 1,120 ft. el.	airtest	4 hrs.	100 gpm	35 ft.	700	-170	6 in.	-90	200	100 gpm		Drawdown is expected to lower the water table to an elevation 170 feet below the BOH. It is anticipated that 710 feet additional drilling is needed to maintain current water supply.
351	T.11N., R.45E.	750	570	740	675	overburden to 748 ft. el.	airtest	1 hr.	400 gpm	NA	700	130		90	0	400 gpm		Drawdown is expected to lower the water table to an elevation 130 feet above the BOH. It is anticipated that no additional drilling is needed to maintain current water supply.
357	T.11N., R.45E.	800	720	740	747	overburden to 788 ft. el.	pump	6 hrs.	30 gpm	4 ft.	700	-20	8 in.	-40	150	180 gpm	230 ft.	Drawdown is expected to lower the water table to an elevation 20 feet below the BOH. It is anticipated that 150 feet additional drilling is needed to maintain current water supply.

Note: Negative DWSE-BOH indicates that the river would be drawn down below the bottom of the well.

Table P2. Summary of New Pump Characteristics

Well No.	Flow (gpm)	Lift (ft)	Pump Efficiency (assumed)	New Pumps Calculated Horsepower	New Pumps Nominal Horsepower
97	1,200	390	0.8	235	250
195	550	480	0.8	124	1,500
95	1,000	390	0.8	196	200
99	250	350	0.8	46	50
182	1,080	276	0.8	173	200
208	30	525	0.8	7	10
215	300	170	0.8	38	40
239	1,000	365	0.8	188	200
255	170	375	0.8	33	40
279	450	265	0.8	70	75
294	4	365	0.8	1	1
300	300	215	0.8	42	50
322	150	240	0.8	22	25
324	34	350	0.8	6	10
357	.180	230	0.8	26	25

Note: For wells pumping less than 25 gpm, it is assumed that 60 psi (139 ft) is needed beyond the ground surface. For wells 25 gpm or greater, it is assumed that 100 psi (231 ft) is needed beyond the ground surface.

Annex Q

Potlatch Corporation Water Intake Modification Plan

Annex Q: Potlatch Corporation Water Intake Modification Plan

Q.1 General

The concepts and costs for this water intake modification plan derive from a separate report prepared for the Corps by Thomas, Dean & Hoskins, Inc., titled *Snake River Drawdown Feasibility Study for: Primary Plant Water Intake – Potlatch Corporation* (TDH 1998a). Modifications described here are not considered as part of the project implementation costs. The plan and costs were developed for economic evaluations of local, regional and national impacts..

The Lewiston, Idaho, division of Potlatch Corporation imports wood and wood byproducts (wood chips) and manufactures and supplies wood, paper, and consumer products to the Northwest. The Clearwater River, which also forms part of the upper end of the Lower Granite Reservoir, supplies all water to the plant. There are currently two independent water intake structures at Potlatch located in the reservoir portion of the Clearwater River:

1) The primary plant water intake
2) An experimental fish hatchery intake (currently unused).

Potlatch is mainly concerned with the impact of the possible drawdown of the lower Snake River reservoirs on the primary plant water intake. Consequently, this study team examined only the need to improve the primary intake structure. The principal effect of drawdown on the existing system would be a decrease in the low-water water surface elevation.

The primary plant water intake consists of the following:
1) A 9-meter (m) (30-foot) deep wet well structure that is a concrete vault roughly 8 m (25 feet) square, constructed immediately adjacent to the levee along the Clearwater River.
2) The wet well inlet with bar screen inlets located above the bottom of the wet well. When lower stream elevations occur after drawdown, the bar screen inlets would be above the water surface.
3) Traveling screens that provide filtration of large material.
4) Five pumps with 200-horsepower motors located at the top of the vault structure.
5) A 60-millimeter (mm) (24-inch) or 762-mm (30-inch) supply pipeline to the treatment plant.

Based on information from Potlatch Corporation records, the elevation at the bottom of the wet well is approximately 220 m (723 feet). The lowest elevation along the stream bottom adjacent to the wet well is 219 m (719 feet). The current operating pool level varies between 223 m (733 feet) and 228 m (748 feet).

According to Potlatch, the existing water intake system is operating well and serves the facility adequately. One isolated hydrologic event, the January flood of 1996, did affect the facility. During this flood event the extreme river flow deposited enough sediment adjacent to the water intake structure to force a plant shutdown. The condition was temporary, and the plant resumed operation within a week. Improvements outlined in this annex would not be required under current operation, but would be required if the proposed drawdown occurs.

Q.2 Standards

Q.2.1 Relevant Codes and Standards

Potable water supply is regulated by the U. S. Environmental Protection Agency under the regulations stipulated in Title 40, Chapter I, Subchapter D - "Water Programs" of the *Code of Federal Regulations*. Since several endangered species within the Snake and Clearwater rivers might be affected by construction operations, all work within the rivers is also regulated by the National Marine Fisheries Service as stipulated in Title 50 of the *Code of Federal Regulations*.

Q.2.2 Design Criteria

Primary Plant Water Intake Requirements are as follows:

- Average daily demand: 9-12 million liters (35-45 million gallons) per day (source: Potlatch)
- Peak flow: 2.0 cubic meters per second (m^3/s) (71.3 cubic feet per second [cfs]) (source: Potlatch)

Postdrawdown Stream Characteristics are as follows:

- Minimum stream bottom elevation (adjacent to wet well): 219 m (719 feet) (source: Potlatch)
- Low water elevation (at 28 m^3/s [1000 cfs]): 222 m (728.4 feet) (source: Corps)
- 100-year high water elevation (at 3,682 m^3/s [130,000 cfs]): 227 m (744 feet) (source: Corps)

Q.3 Intake Modifications

Q.3.1 Options

Because there are many unknowns concerning the effects of drawdown, such as river high-water elevation and velocity and stream bottom elevations, the options included in this report were selected based on a number of assumptions as well as engineering judgment.

The existing water intake structure is shown in Figure Q1. The principal effect of drawdown on the existing system would be a decrease in the low-water elevation. The actual low-water elevation after drawdown would be below the existing wet well bar screen inlet. This study team assumed that the existing wet well structure could be used, and that it is deep enough to collect water from the low stream water condition after drawdown. Two options were considered:

1) Install new bar screens in wet well.
This option allows operation of the existing intake structure. A new inlet would be cut in the existing wet well, and a new bar screen would be attached. The existing bar screen would be removed and the existing inlet sealed. The primary disadvantage to this option is that significant channel change would be required near the wet well. This channel would be subject to significant sedimentation, requiring regular maintenance.

2) Install new screened inlets in stream channel.
This is a relatively maintenance-free option. Screened inlets constructed within the natural stream channel would collect water from the Clearwater River and transport the water to the existing wet well via collector pipes. Several types of screened inlets are available. A common inlet structure consists of Johnson Screens constructed on vertical pipe sections that connect to the wet well intake pipes (see Figure Q2). Johnson Screens are continuously slotted screens constructed in a tubular shape. The screens are easy to install, easy to maintain, and are efficient to use (high percentage of open area over total surface area).

The study team assumed that Johnson Screens would be used and estimated that three T-54 screens would be required to meet peak plant demand. To allow one screen to be out of operation without affecting performance, one additional screen is recommended. Two screens of the four screens would be constructed on each conductor pipe. This configuration would allow the plant to operate at roughly 70 percent of peak demand should one conductor pipe be removed from service for maintenance.

The study team assumed that the foundation conditions would permit the construction of a suitable foundation for the intake screens and that it could be located and protected so that scour would be prevented.

Potlatch Corporation uses on-site wells for potable water supply within the plant. Therefore, improvements to the primary plant water intake system would only affect processing operations. Potlatch has indicated that the plant has a permit to use a temporary water intake system that is installed with a crane adjacent to the primary intake when necessary. This temporary system can supply up to 0.5 million liters (2 million gallons) per day.

Q.3.2 Construction Methods

The study team selected the installation of screened intakes in the stream channel as the most reliable modification. This option is the least susceptible to problems associated with the accumulation of sediments. There are two primary construction methods used for this type of in-water construction.

A common method is to surround the construction area with a sheetpile cofferdam system that allows the construction area to be dewatered. The installation of the pipeline and the four screened intakes can take place in relatively dry conditions. After completion of the installation, the sheetpile is removed. This work would most likely proceed after drawdown occurs so the head pressure on the sheetpile and subsequent seepage into the construction area is minimized. This coffercell system, similar to that used for bridge pier construction, is directly applicable to construction of the river intakes. Dewatering of the trench segment presents some stability concerns. The major assumption is that there is sufficient overburden so that driven sheetpile segments remain stable without cross-bracing.

Underwater construction could be used only if construction took place before drawdown occurred. This is because stream velocities would be much higher after drawdown, making construction with divers difficult. Work would be staged on a floating work platform using a derrick crane to excavate and place material. Elements would be prefabricated on deck. In-water installation by divers would be minimized. Underwater construction for this installation would create turbidity in the water caused by excavation and backfill of the trench.

The study team selected cofferdam construction rather than underwater construction for installation of the screened inlets. The construction method for modifying the Potlatch primary plant water intake system using Johnson Screens is summarized as follows:

1) Cofferdams
   Cofferdams would be constructed around the proposed Johnson Screen locations shown in Figure Q2. Due to the location of the screens in the middle of the river, only one cofferdam installation would be required, and stream flow would be diverted to the north side of the river. Sheetpile cofferdams are proposed.

2) Trench Excavation
   Trenches would be constructed using open-cut trench methods. Assuming two 1,067-mm (42-inch) conductor pipes would be installed, the minimum trench width would be 2 m (5 feet 6 inches). It is

assumed that the pipe would be installed above 305 mm (12 inches) of pipe bedding and that 914 mm (36 inches) of cover would be required above the pipe. Therefore, the total trench depth would be 2 m (7 feet 6 inches).

3) Pipeline Installation
Conductor pipes would be 1,067-mm (42-inch) ductile iron. Two conductor pipes are required, each 76 m (250 feet) in length. Vertical pipe supports for the Johnson Screens would be 1,067-mm (42-inch) ductile iron pipe.

4) Air Backwash Piping
Conduits for the backwash system would consist of 102-mm (4-inch) ductile iron pipes between the pump house (above the wet well) and the Johnson Screens. These conduits would be installed within the conductor pipe trenches. Backwash piping would be installed up to the wet well pump location where they could be accessed for maintenance. A compressor could be brought on site and connected to the piping when screen cleaning is necessary.

5) Trench Backfill
Trenches would be backfilled with granular borrow from the top of the pipe bedding to the existing ground surface. Sources for the backfill material are available within the local area, minimizing long haul charges.

6) Johnson Screen Installation
The four Johnson Screens are each 1,372-mm- (54-inch-) diameter tubular structures. All screens would be installed on 1,067-mm (42-inch) vertical pipe supports in line with the river flow.

7) Channel Armor
Existing channel slopes and the stream bottom would be stabilized with heavy riprap. The study team estimated that the material size would be between 457 mm (18 inches) and 914 mm (36 inches) in size (D_{50} = 610 mm [24 inches]). Riprap would be placed a distance of 30 m (100 feet) upstream and downstream of each pipeline. The team assumed that riprap would be hauled from a nearby quarry and that some haul costs would be incurred.

8) Remove Cofferdams
Once the pipelines and screens were tested and accepted, the sheetpile cofferdams would be removed, and the system placed in service.

9) Signage
There would need to be signs installed on either side of the river to warn against dropping anchors through this part of the river. This sign would also need to prohibit boats that displace more than 0.6 m (2 feet) of water from using this river during low stream flow periods of the year.

10) Debris Impact Structure
Three piles 4 m (12 feet) long, driven 1.5 m (5 feet) into the riverbed would support a high strength grate 3 m (10 feet) wide and 2 m (7 feet) high. This structure would protect the Johnson Screens.

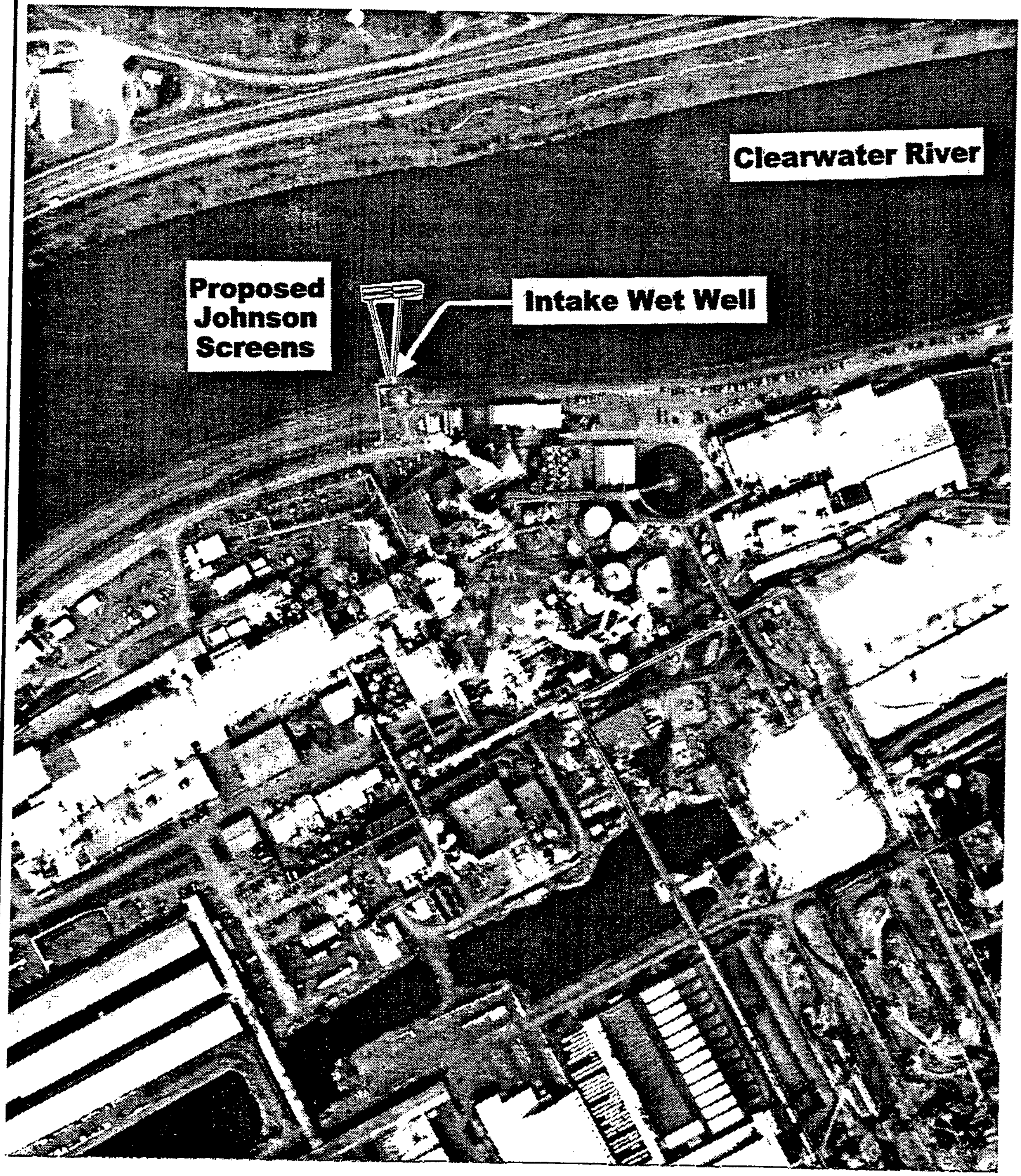
Q.3.3 Construction Materials

1) Pipe: 1,067-mm (42-inch) outside diameter (OD), class 51 ductile iron (wall thickness = 13 mm [0.53 inches])
2) Pipe Fittings: Mechanical joints
3) Pipe Coating: Cement-mortar interior lining.

Q.4 Schedule

There is no significant variation in process water consumption by the Potlatch plant throughout the year. The ideal time for improvements to the water intake structure would be during an annual plant shutdown. Potlatch recommends that screen and pipe installations be made prior to shutdown. During the shutdown, three days would be available to make connections to the wet well without affecting plant operations.

Cofferdams represent a significant portion of the construction cost for the project. The ideal construction window would occur between September and December when stream flows are minimal. This would minimize the size of cofferdam used.



Thomas, Dean & Hoskins, Inc.

SHEET MAIN SCALE

US Army Corps of Engineers Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
POTLATCH CORP. PRIMARY PLANT WATER INTAKE
SITE PLANS

Figure: Q1

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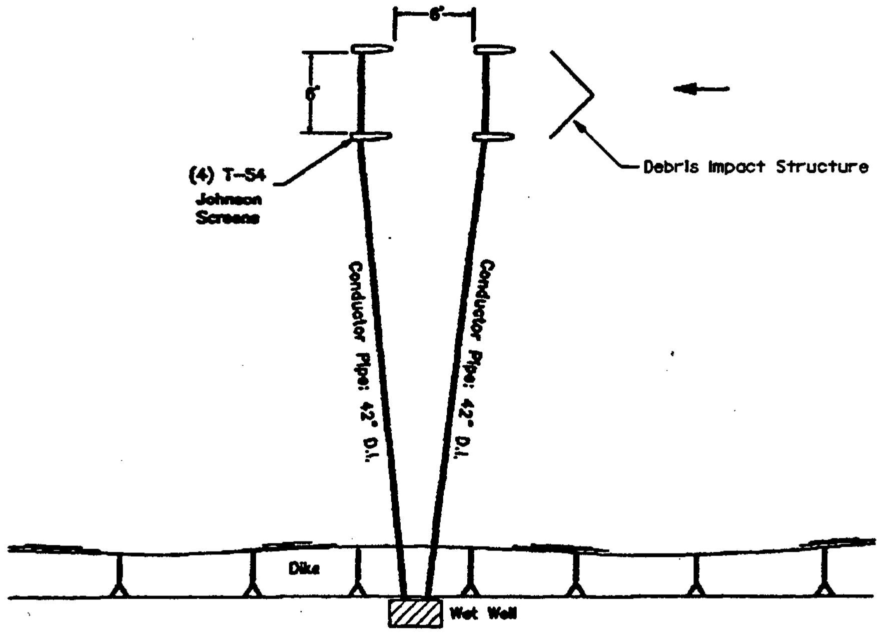
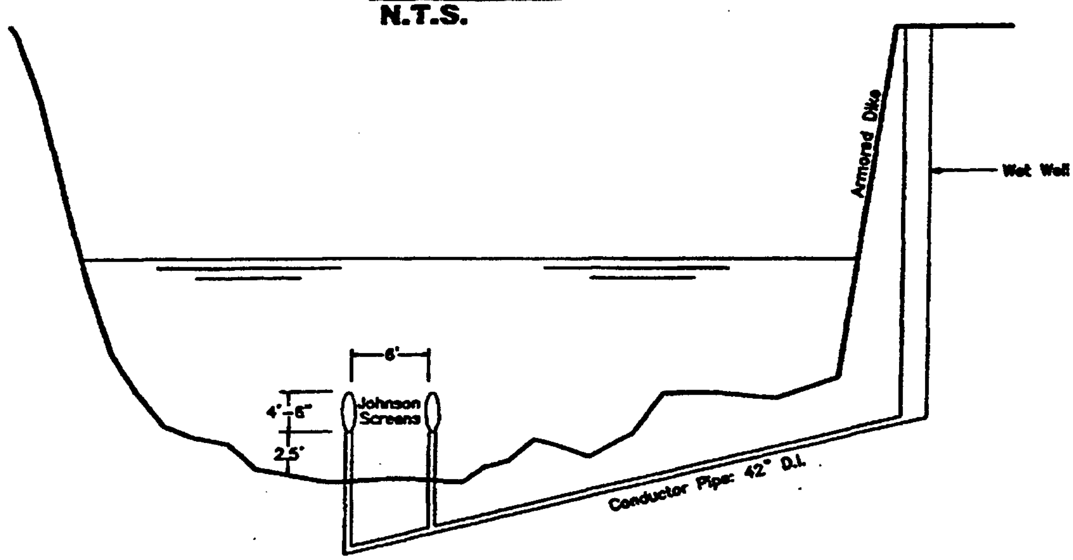
CLEARWATER RIVER



PLAN VIEW
N.T.S.



RIVER CROSS SECTION
N.T.S.

Thomas, Dean & Hoskins, Inc. 10/02/98

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
POTLATCH CORP. PRIMARY PLANT WATER INTAKE
JOHNSON SCREEN INSTALLATIONS

Figure:
Q2

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Annex R

Other River Structures Modification Plan

Annex R: Other River Structures Modification Plan

R.1 General

The Modification Plan for several water intake and effluent structures that are not appropriately included in other annexes is described in this annex. They are all privately-owned or municipally-owned structures. It should be noted that the modifications proposed herein are non-Federal modifications and it is unknown at this time whether Congress will fund them. The water intake and effluent structures are:

Atlas Water Intake
Clarkston Golf Course Water Intake
Lewiston Golf Course Water Intake
Asotin Sewage Outfall
Clarkston Sewage Outfall
Lewiston Sewage Outfall

Water intakes for municipal water supply are supplied by sources not directly impacted by drawdown. Surface water intakes are located upstream of the Lower Granite pool on the Clearwater River or utilize groundwater from wells.

R.2 Atlas Sand and Rock Water Intake

Atlas Sand and Rock produces a variety of rock products for concrete, asphalt, and other construction applications. Water is utilized for a variety of process and housekeeping uses including, aggregate washing, dust control, and equipment maintenance. Water for non-potable use is pumped from the Snake River using a single 100 hp vertical turbine pump having a peak capacity of 1050 gpm.

Drawdown of the Lower Granite Reservoir will result in a seasonal varying water surface elevation. A system and facility was conceived as one option to provide a reliable water supply. This system is a moveable system that allows the pump to be located as necessary to accommodate a changing river level as well as repositioning the intake in the event that sediment accumulation creates problems.

The system is a trailer mounted pump with a flexible suction intake. The facility includes extension of supply pipe, electrical service, and a storage building to house the electrical panels, spare equipment, and maintenance items.

R.3 Clarkston Golf Course Water Intake

The Clarkston City Golf Course is an 18-hole facility owned and operated by the City of Clarkston. Water is utilized for irrigation of certain areas of the golf course. Water for this use is pumped from the Snake River using a single 10 hp centrifugal pump having a peak capacity of approximately 100 gpm.

Drawdown of the Lower Granite Reservoir will result in a seasonal varying water surface elevation. A system and facility was conceived as one option to provide a reliable water supply. This system is a moveable system that allows the pump to be located as necessary to accommodate a changing river level as well as repositioning the intake in the event that sediment accumulation creates problems.

The system is a trailer mounted pump with a flexible suction intake. The facility includes extension of supply pipe, electrical service, and a storage building to house the electrical panels, spare equipment, and maintenance items.

R.1 Lewiston Golf Course Water Intake

The Lewiston City Golf Course is an 18-hole facility owned and operated by the City of Lewiston. Water is utilized for irrigation of certain areas of the golf course. Water for this use is pumped from the Snake River using a single 60 hp centrifugal pump having a peak capacity of approximately 450 gpm.

Drawdown of the Lower Granite Reservoir will result in a seasonal varying water surface elevation. A system and facility was conceived as one option to provide a reliable water supply. This system is a moveable system that allows the pump to be located as necessary to accommodate a changing river level as well as repositioning the intake in the event that sediment accumulation creates problems.

The system is a trailer mounted pump with a flexible suction intake. The facility includes extension of supply pipe, electrical service, and a storage building to house the electrical panels, spare equipment, and maintenance items.

R.2 Asotin Sewage Outfall

The City of Asotin wastewater treatment plant discharges treated water into the Snake River at approximately river mile 145. Outfall pipe modifications performed in 1975 provide a new outfall line and diffuser discharge. Diffuser port elevations are approximately 721 feet. Drawdown water surface elevations are estimated to range between 730 and 756 feet for flows between 566 m^3/s (20,000 cfs) and 9,000 m^3/s (320,000 cfs), respectively. It is not anticipated that possible future drawdown of the Lower Granite reservoir will require any physical changes to the diffuser configuration or orientation. No system or process costs were estimated for this facility.

R.3 Clarkston Sewage Outfall

The City of Clarkston wastewater treatment plant discharges treated wastewater into the Snake River at approximately river mile 138. Outfall pipe modifications performed in 1997 replaced the 12-inch outfall pipe with a 16-inch outfall pipe and extended the outfall diffuser over 500 feet. The resulting location of the diffuser is in the original channel of the Snake River. Diffuser port elevations are approximately 697 feet. Drawdown water surface elevations are estimated to range between 708 and 729 feet for flows between 566 m^3/s (20,000 cfs) and 9,000 m^3/s (320,000 cfs), respectively. It is not anticipated that possible future drawdown of the Lower Granite reservoir will require any physical changes to the diffuser configuration or orientation. No system or process costs were estimated for this facility.

R.4 Lewiston Sewage Outfall

The City of Lewiston wastewater treatment plant discharges treated waste water into the Clearwater River at approximately river mile 1. This location is approximately 1 mile upstream from the Clearwater River confluence with the Snake River. The effluent diffuser consists of a buried 914-mm (36-inch) diameter pipe that extends approximately 40 feet into the river. The invert elevation of the pipeline is 703 feet. The diffuser section is a series of gradually smaller concrete-encased pipe sections ranging from 889 mm to 610 mm (35 inches to 24 inches) in diameter that further extend into the river. The 14 diffuser ports are spaced at 2-m (8-foot) intervals at elevation 707 feet (est.). Drawdown water surface elevations are estimated to range between 712 and 735 feet for flows between 566 m^3/s (20,000 cfs) and 9,000 m^3/s (320,000 cfs), respectively. Several facility modifications are currently under consideration to

improve the dilution zone conditions for current reservoir operation. Further studies are necessary to determine whether a natural river operation would necessitate diffuser modifications.

Annex S

Potlatch Corporation Effluent Diffuser Modification Plan

Annex S: Potlatch Corporation Effluent Diffuser Modification Plan

S.1 General

The concepts and costs for this water intake modification plan derive from a separate report prepared for the Corps by Thomas, Dean & Hoskins, Inc., titled *Snake River Drawdown Feasibility Study for: Plant Effluent Diffuser – Potlatch Corporation* (TDH 1998b). Modifications describe here are not considered as part of the project implementation costs. The plan and costs were developed for economic evaluations of local, regional and national impacts.

The Lewiston, Idaho, division of Potlatch Corporation imports wood and wood byproducts (wood chips) and manufactures and supplies wood, paper, and consumer products to the Northwest. Treated effluent from the plant is conveyed to its discharge location through a 1,219-millimeter (mm) (48-inch) effluent pipeline west of the plant. The effluent pipeline enters the Clearwater River at approximately river kilometer 0.85 (river mile 0.53). From its entrance into the Clearwater River, the 1,219-mm (48-inch) effluent line then runs west to the confluence with the Snake River, where a 122-meter- (m) (400-foot-) long diffuser section is located.

The effluent diffuser consists of a 1,219-mm (48-inch) pipeline fitted with 102-mm (4-inch) nozzles located at the top of the pipe. The 102-mm (4-inch) nozzles are connected to 76-mm (3-inch) polyethylene pipes that discharge effluent into the stream. Diffusers are spaced 1.5 m (5 feet) center to center.

During the 1992 experimental drawdown (Corps 1992), the top portion of the polyethylene diffusers were exposed. It is clear that a proposed drawdown of the four lower Snake River reservoirs would mandate the relocation of the effluent diffuser to a deeper area within the river.

According to Potlatch, the existing effluent pipeline and diffuser system is operating adequately. An underwater inspection was conducted a year prior to this study and revealed that only minor improvements were required to replace a few polyethylene diffusers. Improvements outlined in this report would not be required under current plant operation, but would be required if the proposed drawdown occurs.

Currently, there is much debate on the issue of temperature of the river water and the consequent water quality. There has been discussion on whether the Potlatch effluent affects the temperature regime and whether water treatment to control effluent temperature is necessary. The added effects of drawdown provide further concerns. This annex only address a replacement system to relocate the existing effluent diffuser. Treatment systems for the water are beyond the scope of this report but future circumstances may required water treatment systems to be added to this modification plan.

S.2 Standards

S.2.1 Relevant Codes and Standards

Effluent discharge is regulated by the U.S. Environmental Protection Agency under the regulations stipulated in Title 40, Chapter I, Subchapter D - "Water Programs" of the *Code of Federal Regulations.* Since several endangered species within the Snake and Clearwater rivers might be affected by

construction operations, all work within the rivers is also regulated by the National Marine Fisheries Service as stipulated in Title 50 of the *Code of Federal Regulations.*

S.2.2 Design Criteria

1) Plant Effluent Requirements:
   - Average daily discharge: 160-200 million liters (35-45 million gallons) per day (source: Potlatch)
   - Peak flow: 2.0 cubic meters per second (m^3/s) (71.3 cubic feet per second [cfs]) (source: Potlatch)

2) Postdrawdown Stream Characteristics:
   - Minimum stream bottom elevation (at proposed diffuser location): 212 m (694 feet) (source: Corps)
   - Low water elevation (at 566 m^3/s [20,000 cfs]): 216 m (709 feet) (source: Corps)
   - 100-year high water elevation (at 9,062 m^3/s [320,000 cfs]): 220 m (723 feet) (source: Corps)

S.3 Diffuser Modifications

S.3.1 Options

Because there are many unknowns concerning the effects of drawdown, such as river high-water elevation and velocity and stream bottom elevations, the options included in this report were selected based on a number of assumptions as well as engineering judgment.

The existing effluent pipeline and diffuser is shown in Figure S1. Based on streambed profiles supplied by the Corps and streambed contours provided by Potlatch, it appears that the existing diffuser is located on a shelf within the Snake River.

Assuming the stream bottom profiles are accurate, the stream bottom is significantly deeper (by about 0.9 m [3 feet]) just 366 m (1200 feet) downstream of the existing effluent diffuser location. The proposed location for the new effluent diffuser is within this deeper pool along the Snake River. Because the existing facility is performing adequately, the existing effluent pipeline up to the first angle point would be used in this modification plan. The new diffuser would be designed with the same length and geometry as the existing diffuser, but would be installed perpendicular to the river flow at its new location. Future design should include an evaluation of sediment erosion to determine the stable depth at which to set the pipe. This is to prevent erosion of the foundation under the pipeline or adverse hydraulic conditions in the river.

Manholes would be constructed at pipeline angles and at the terminus of the new diffuser. The manhole at the angle point consists of 1,219-mm (48-inch) flange fittings with a short section of 1,219-mm (48-inch) vertical pipe. The end of the vertical pipe would be fitted with a blind flange. The terminus manhole would consist of 1,219-mm (48-inch) flange fitting with a 914-mm (36-inch) vertical pipe and a special diffuser connection.

The effective operation of the diffuser requires that it be continually submerged. This requirement forces the installation of the new pipeline and diffusers to be completed prior to initiating drawdown. Sediment deposition in this region has resulted in a significant accumulation of sediment that must be excavated to install the diffuser pipeline. Most of this accumulated sediment on the river bed is predicted to be eroded away during the first year drawdown occurs. See Figure S2 for a cross section of the installed effluent diffuser.

Since river flows are not affected by drawdown, the study team assumed that no change in the dilution of the effluent would occur by postdrawdown stream flow. If it is determined that additional treatment would be required, the estimated construction costs noted in this report would not adequately cover the additional costs for this treatment.

Changes to the effluent diffuser to prepare for drawdown conditions will require consultation with various regulatory agencies with respect to effects to endangered and threatened species. Issues regarding mixing of effluent with river water may require additional systemic modifications to be made to guarantee compliance with permit conditions. Such modifications could result in additional capital costs ranging up to $50 million should a complete wastewater treatment facility be necessary.

S.3.2 Construction Methods

There are two primary construction methods used for this type of in-water construction.

A common method is to surround the construction area with a sheetpile cofferdam system that allows the construction area to be dewatered. The installation of the pipeline section and the diffuser section can take place in relatively dry conditions. After completion of the installation, the sheetpile is removed.

Underwater construction would include use of a barge with a clamshell excavator to excavate the trench. The upstream part of the trench would have cofferdams installed to prevent trench wall caving. There are two options available for installing the pipe underwater. One option is to preconstruct up to three 12-m (40-foot) lengths of pipe on board a barge, cap the pipe, float it to the installation location, sink it, uncap it, and connect it underwater to the existing pipe. Another option is to preconstruct five or six lengths of 12-m (40-foot) pipe on shore, then cap them and float them out to the installation location. The pipe is then sunk, uncapped, and connected to the existing pipe. This procedure would use four divers, two diver tenders, and a crew to support the excavation and dry land connections of pipe. It would be typically less expensive to work off the shore connecting the pipe rather than on the barge, if possible. The cost for these procedures is comparable to using cofferdams to provide for all dry construction.

The study team selected cofferdam construction rather than underwater construction for installation of the pipeline sections. The primary reason was that underwater construction created turbidity in the water caused by excavation and backfill of the trench.

The construction method for modifying the Potlatch plant effluent diffuser system is summarized as follows:

1) Cofferdams
   Cofferdams would be constructed in two phases. The first phase would consist of cofferdams around the effluent pipeline between the new diffuser and the connection to the existing effluent pipeline. Once the pipeline installation within this area is complete, the cofferdams would be removed and cofferdams would be constructed around the proposed effluent diffuser. For the purposes of this study, sheetpile cofferdams are proposed.

2) Trench Excavation
   Trenches would be constructed using open-cut trench methods. Assuming 1,219-mm (48-inch) pipelines would be installed, the minimum trench width would be 2 m (6 feet). This study team assumed that the pipe would be installed above 305 mm (12 inches) of pipe bedding and that 914 mm (36 inches) of cover would be required above the pipe. Therefore, the total trench depth would be 2.4 m (8 feet).

3) Pipeline Installation
Effluent and diffuser pipelines would be 1,219-mm (48-inch) ductile iron. The total length of solid effluent pipe between the existing effluent pipeline and proposed diffuser is 914 m (3,000 feet). An additional 122 m (400 feet) of perforated ductile iron pipe would be required for the diffuser. Perforations would be 102-mm (4-inch) drilled holes, threaded to accept the 102-mm (4-inch) ductile iron diffuser pipes. Then 76-mm (3-inch) polyethylene diffusers would be attached to the end of the ductile iron diffuser segments. Approximately 457 mm (18 inches) of polyethylene diffuser would be exposed above the stream bottom.

4) Concrete Fill of Abandoned Pipe and Diffuser
A high slump concrete would be pumped into the abandoned pipe and diffuser. Existing, exposed diffuser pipes would be cutoff at ground level and removed from the site.

5) Trench Backfill
Trenches would be backfilled with granular borrow from the top of the pipe bedding to the existing ground surface. Sources for the backfill material are available within the local area, minimizing long haul charges.

6) Streambed Armor
The existing stream bottom over the new pipeline would be lined with heavy riprap to preclude scour. The study team estimated that the material size would be between 457 mm (18 inches) and 914 mm (36 inches) in size (D_{50} = 610 mm [24 inches]). Riprap would be placed a distance of 30 m (100 feet) upstream and downstream of each pipeline. The team assumed that riprap would be hauled from a nearby quarry, and that some haul costs would be incurred.

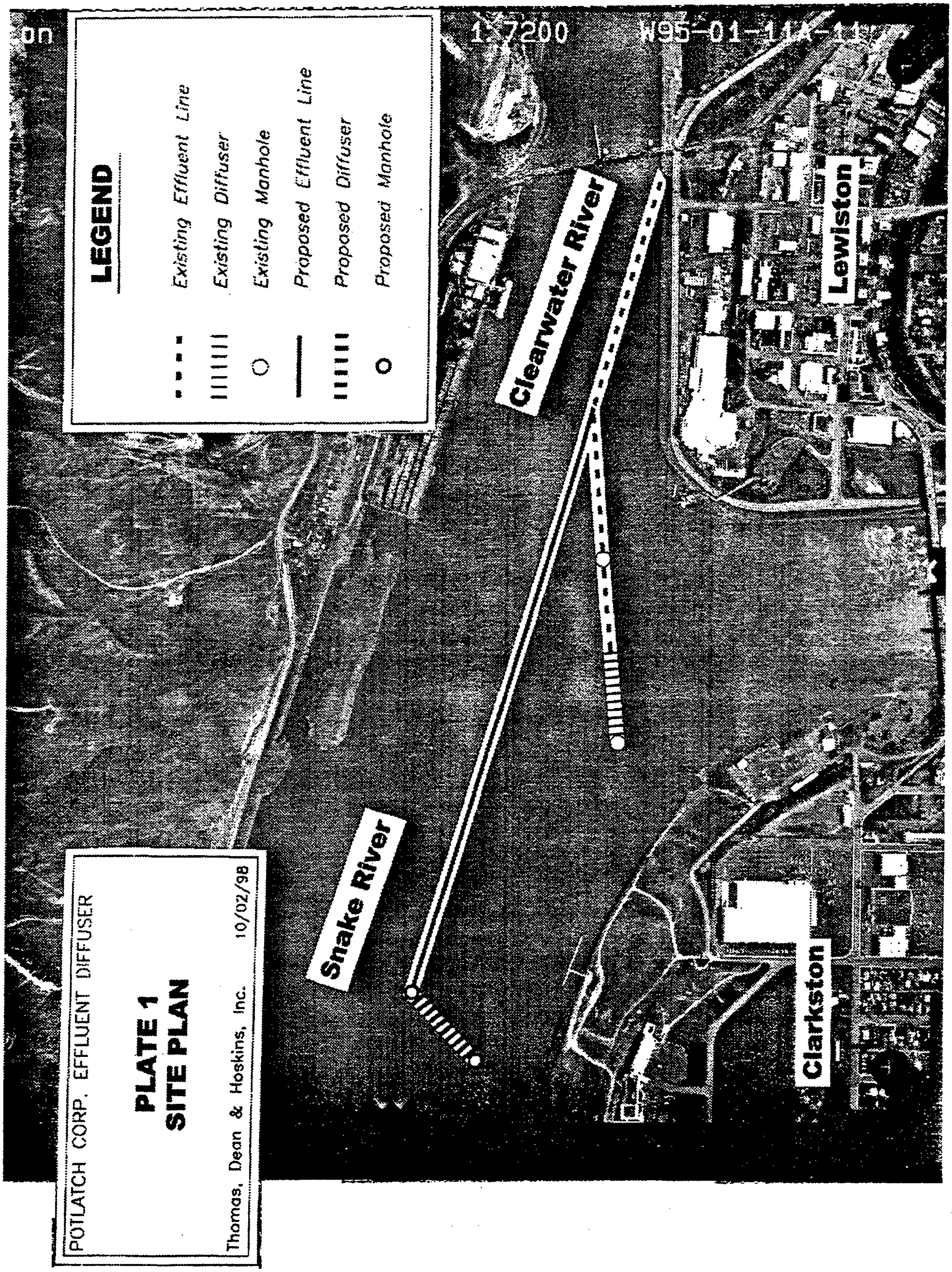
7) Remove Cofferdams
Once the diffuser installation is tested and accepted, the sheetpile cofferdams would be removed, and the system placed in service.

S.3.3 Construction Materials

1) Pipe: 1,067-mm (42-inch) outside diameter (OD), class 51 ductile iron (wall thickness = 13.5 mm [0.53 inches])
2) Pipe Fittings: Mechanical joints
3) Pipe Coating: Asphaltic lining.

S.4 Schedule

Work would need to be done before drawdown. The ideal construction window would occur between October and November when stream flows are minimal. Working during minimized stream flows would minimize construction costs.



US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

POTLATCH CORP. EFFLUENT DIFFUSER

SITE PLAN

Figure: S1

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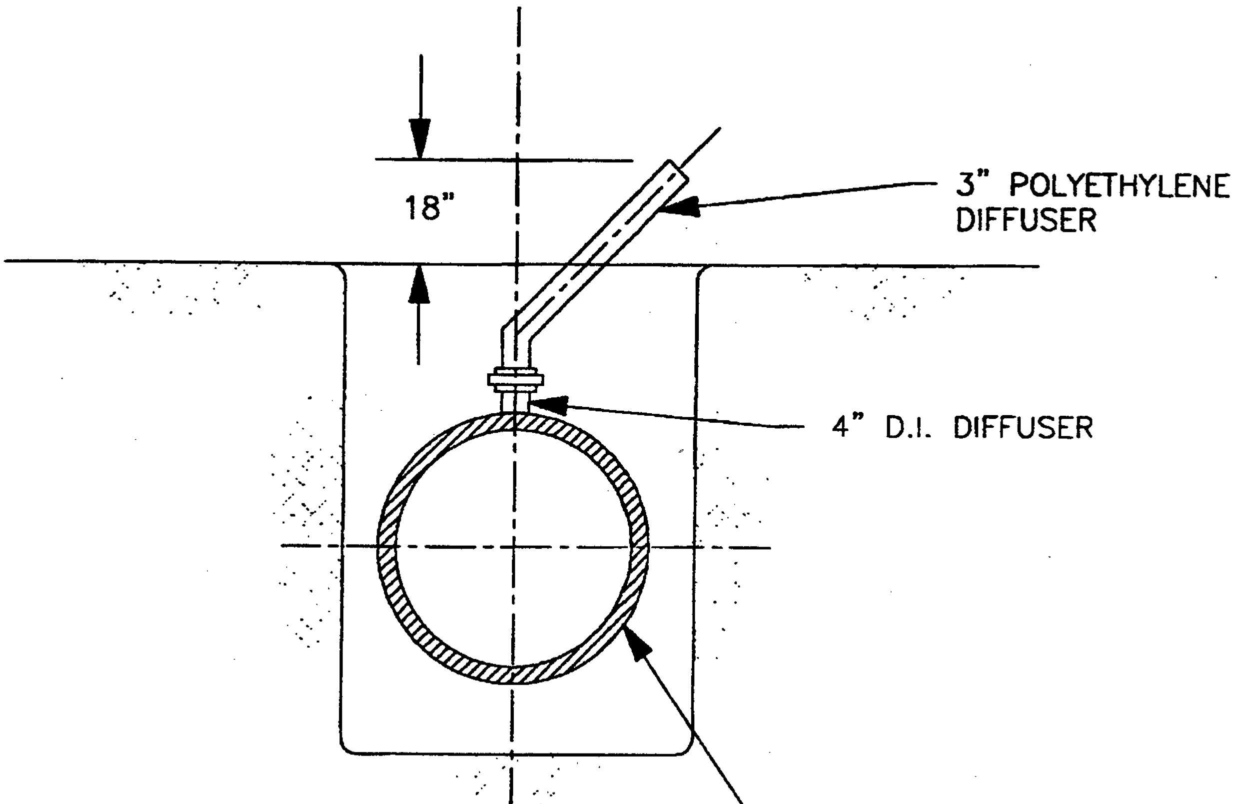

TYPICAL SECTION

Thomas, Dean & Hoskins, Inc.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
POTLATCH CORP. EFFLUENT DIFFUSER
EFFLUENT PIPELINE TRENCH - SECTION

Figure: S2

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Annex T

PG&E Gas Transmission Main Crossings Modification Plan

Annex T: PG&E Gas Transmission Main Crossings Modification Plan

T.1 General

The concepts and costs for this water intake modification plan derive from a separate report prepared for the Corps by Thomas, Dean & Hoskins, Inc., titled *Snake River Drawdown Feasibility Study for: Gas Transmission Main Crossings – PG&E Gas Transmission* (TDH 1998c). Modifications described here are not considered as part of the project implementation costs. The plan and costs were developed for economic evaluations of local, regional and national impacts.

Pacific Gas and Electric (PG&E) Gas Transmission Northwest of Portland, Oregon, serves the Northwest with natural gas. Natural gas is supplied to eastern Washington gas providers via two gas transmission mains, referred to as the "A" Line and the "B" Line. Two Snake River crossings exist near the town of Starbuck, Washington, in Columbia and Whitman Counties, Washington, in Section 33, Township 13 North, Range 37 East, Willamette Meridian. The two crossings lie within the Lower Monumental Dam reservoir approximately 46 meters (m) (150 feet) apart.

According to PG&E, the "A" Line was constructed in the 1950s and the "B" Line was constructed in the 1980s. Each gas main is 914-millimeter (mm) (36-inch) diameter steel pipe with thickness varying from 9.65 mm (0.380 inches) to 15.9 mm (0.625 inches). Typically, the pipe thickness increases in sensitive areas, including river crossings. The existing pipes were installed below the stream bed using open cut trenching methods and bedded and covered with granular backfill. The average trench depth is 3 m (10 feet). Underwater pipe installations were made by encasing the pipe with concrete to prevent possible buoyancy.

T.2 Standards

T.2.1 Relevant Codes and Standards

The U. S. Department of Transportation regulates natural gas transmission mains constructed within the United States. Design, construction, and maintenance requirements are specified in Title 49, Part 192 - "Transportation of Natural and Other Gas by Pipeline: Minimum Federal Safety Standards" of the *Code of Federal Regulations.* Additional requirements are specified in American National Standards Institute/National Fire Protection Association 58 and 59, "Standard for the Storage and Handling of Liquefied Petroleum."

T.2.2 Design Criteria

For the two Snake River crossings, the pipeline is within a Class 1 location. A Class 1 location is any area 1.6 kilometers (1 mile) in length and 201 m (220 yards) in width, measured on either side of the centerline of the pipeline, within which there are 10 or fewer buildings intended for human occupancy.

Maximum allowable operating pressure is 6.3×10^3 Pascal (Pa) (911 pounds per square inch, gage pressure [psig].

Postdrawdown stream characteristics are as follows:

1) Gas Pipe "A":
   - Minimum stream bottom elevation (at proposed location): 146.2 m (479.8 feet) (source: Corps)

- Low water elevation (at 566 cubic meters per second [m^3/s] [20,000 cubic feet per second (cfs)]): 149 m (490 feet) (source: Corps)
- 100-year high water elevation (at 9,062 m^3/s [320,000 cfs]): 156 m (513 feet) (source: Corps)

2) Gas Pipe "B":
   - Minimum stream bottom elevation (at proposed location): 146 m (479 feet) (source: Corps)
   - Low water elevation (at 566 m^3/s [20,000 cfs]): 149 m (489 feet) (source: Corps)
   - 100-year high water elevation (at 9,062 m^3/s [320,000 cfs]): 156 m (512 feet) (source: Corps)

T.3 Pipeline Modifications

T.3.1 Options

Because there are many unknowns concerning the effects of drawdown, such as river high-water elevation and velocity and stream bottom elevations, the options included in this report were selected for the "worst-case" scenario, which is total reconstruction of the utility within the area of impact.

The existing natural gas transmission main locations are shown in Figure T1. Effects on the transmission line associated with the drawdown may include the following:

- Significant variations in high water elevation
- Increased stream velocities
- Possible scour and/or sedimentation of the streambed.

Measures to mitigate the potential effects could include construction of two new pipelines parallel to the existing mains. It is feasible to construct the new mains immediately adjacent to the existing pipelines (within 25 feet). Based on conversations with PG&E, it may be desirable to the utility company to construct the two new crossings downstream of the existing pipelines. To establish quantities, however, this study team assumed that the pipelines would be constructed adjacent to existing mains.

In addition to constructing two new crossings, it may be necessary to stabilize existing stream banks to preclude scour. The study team assumed that the preferred method for stabilization would include placing riprap along the new pipelines for a distance of 30 m (100 feet) upstream and downstream of the crossings.

T.3.2 Construction Methods

There are two primary construction methods used for this type of in-water construction.

A common method is to surround the construction area with a sheetpile cofferdam system that allows the construction area to be dewatered. The installation of the pipeline section can take place in relatively dry conditions. See Figure T2 for typical cross section of pipe and trench. After completion of the installation, the sheetpile is removed.

Underwater construction would include use of a barge with a clamshell excavator to excavate and backfill the trench. This procedure would use four divers, two diver tenders, and a crew to support the excavation and dry land connections of pipe. Underwater construction could be used only if construction takes place before drawdown occurs. This is because stream velocities would be much higher after drawdown, making construction with divers difficult. Higher stream velocities might apply undue stresses that could break preconstructed welds.

The study team selected cofferdam construction rather than underwater construction for installation of the pipeline sections. The primary reason was that underwater construction created turbidity in the water caused by excavation and backfill of the trench.

The construction method for reconstructing the natural gas transmission mains is summarized as follows:

1) Trench Excavation
   A 1.5-m (5-foot) wide by 3-m (10-foot) deep trench would be constructed using open cut trenching methods (trackhoe). Based on the study team's site visit and information from PG&E, the team is confident that rock excavation would be required on the north side of the river. For this analysis, the team assumed that half of each pipeline trench would require rock excavation, which would be accomplished by ripping or blasting. Within the stream, trenching operations would require cofferdams and dewatering equipment. The bottom 305 mm (12 inches) of the trench, in areas where no concrete encasement is proposed, would be bedded with suitable granular bedding material.

2) Pipeline Installation
   A new 914-mm (36-inch) pipeline would be constructed within the open trench. The "A" Line installation would be 518 m (1,700 feet) in length, and the "B" Line would be 472 m (1,550 feet) in length.

3) Concrete Encasement
   Any pipe installed below the 100-year high water elevation would require concrete encasement. The primary purpose of the encasement is to preclude flotation. Based on the study team's analysis, the minimum depth of concrete, assuming a 1.5-m (5-foot) wide trench, is 1.5 m (5 feet) to provide a factor of safety of 1.25 against buoyancy. Because the natural stream width is unknown, the study team assumed that 80 percent of the replaced pipeline would require encasement. The total length required, therefore, would be 415 m (1,360 feet) for the "A" Line and 378 m (1,240 feet) for the "B" Line.

4) Trench Backfill
   Trenches would be backfilled with pit run material from the top of the concrete encasement to the existing ground surface. Sources for the backfill material are available within the local area, minimizing long haul charges.

5) Bank Stabilization
   Existing slopes would be stabilized with heavy riprap. The study team estimates that the material size would be between 457 mm (18 inches) and 914 mm (36 inches) in size (D_{50} = 610 mm [24 inches]). Riprap would be placed a distance of 30 m (100 feet) upstream and downstream of each pipeline. It is assumed that riprap would be hauled from a nearby quarry, and that some haul costs would be incurred.

6) Connections to Existing Mains
   Once the previous work is complete, connections could be made to the existing natural gas mains. In order to maintain service to natural gas providers, one main would be taken out of service at a time, leaving the other main in service. Natural gas would be evacuated from the existing pipeline at compressor stations on either side of the river crossing. The existing main would then be cut and fittings welded in place to make connection with the new gas main. Upon completion of the connections on both sides of the river, the new gas main would be placed in service to allow work on the other gas main to commence.

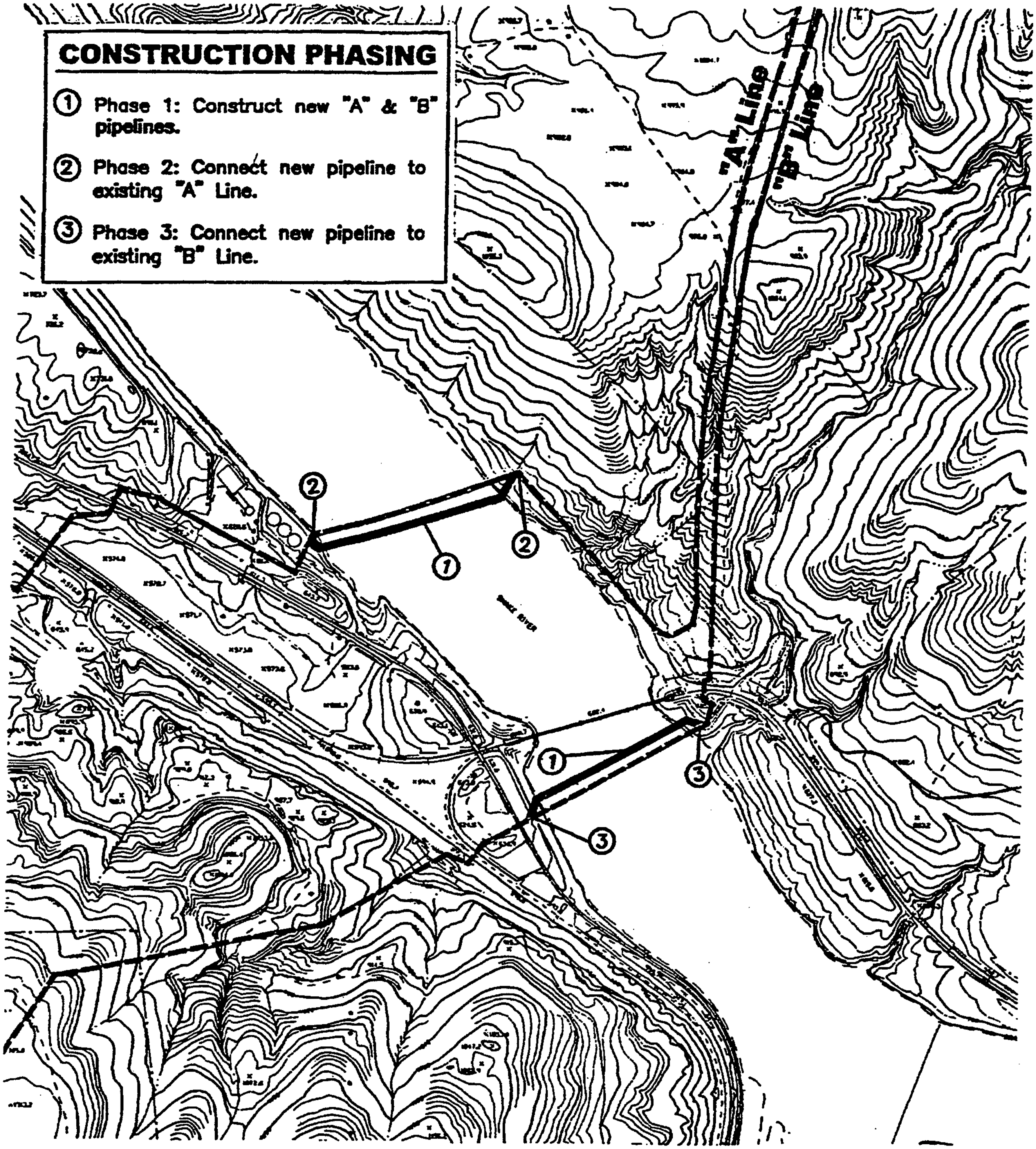
7) Abandon Existing Mains
Existing gas main crossings would be abandoned in place. The existing pipelines would be filled with high slump concrete, and steel caps would be welded on each end.

T.3.3 Construction Materials

1) Pipe: 914 mm (36-inch) outside diameter (OD) x 16.0-mm (0.630-inch) W.T. API 5L x 70 steel
2) Pipe Joints: Welded butt joints
3) Pipe Coating: Fusion bonded epoxy coating
4) Valves: Not required
5) Compressor Stations: Not required
6) Cathodic Protection: Provided by pipe coating

T.4 Schedule

Based on discussions with PG & E, the ideal time for the construction of the new gas pipeline is before drawdown occurs between October and November so that stream velocities are minimal. However, this timeframe may conflict with the in-water construction periods specified by the National Marine Fisheries Service. The actual construction window must be established as part of the environmental scoping process for the project.



LEGEND

Existing 36" Natural Gas Main

Proposed 36" Natural Gas Main

Thomas, Dean & Hoskins, Inc.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

PG&E GAS TRANSMISSION LINES

SITE PLAN

Figure:

T1

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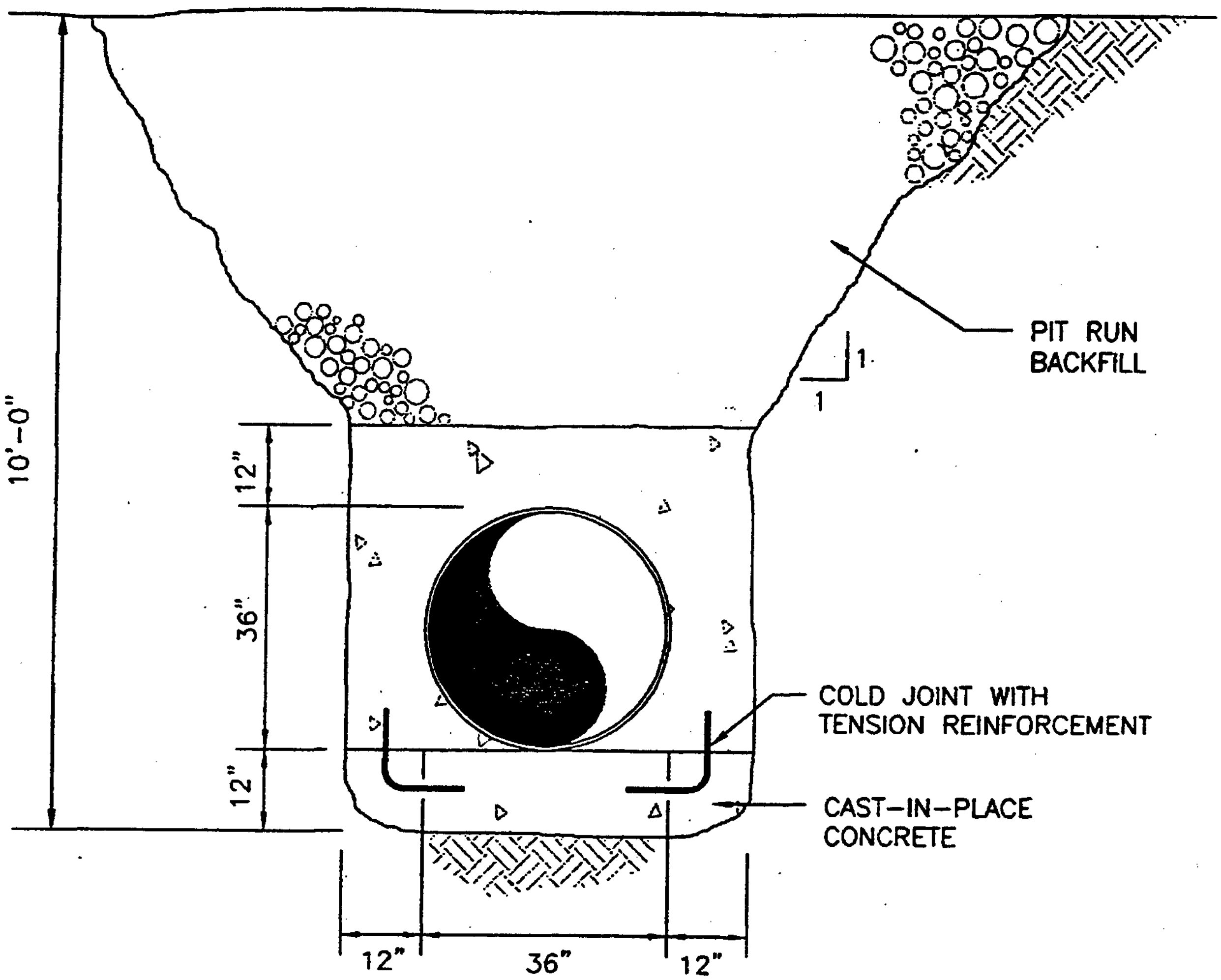

TYPICAL GAS PIPELINE TRENCH SECTION

NOT TO SCALE

Thomas, Dean & Hoskins, Inc.

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY

TYPICAL GAS PIPELINE TRENCH SECTION

Figure: T2

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Annex U

Hydropower Facilities Decommissioning Plan

Annex U: Hydropower Facilities Decommissioning Plan

U.1 General

This portion of the drawdown study details the actions required to decommission the four lower Snake River dams. Decommissioning is defined as preparing the hydropower facilities for suspension of power production, with a possible option of bringing the facilities back on line after an extended outage. The study team for decommissioning these hydropower facilities considered two major actions:

- Breaching the embankment dam and, by constructing levees, permanently channeling the river around the remaining dam structures, and leaving the dam structures in place
- Breaching the embankment dam, temporarily channeling the river around the remaining dam structures, and removing the dam structures from the river.

The study team recommends the first action for reasons discussed in Section 9 of this appendix. That action requires the disposal of most of the hydropower facilities' equipment and on-site waste materials, but leaves the structures in place, yet off-limits, to public access. The options for disposal of all equipment, removal of any hazardous materials, and implementation of facility security are evaluated in this annex. The study team performed a detailed analysis for the Lower Granite Dam and applied that analysis, with appropriate modifications, to Ice Harbor, Little Goose, and Lower Monumental dams to determine the actions and costs for decommissioning all four dams.

U.2 Methodology

Under the first action, two options for disposing of facilities and equipment were identified:

- The Mothball Option involves suspending all operations and maintaining equipment in working condition until a decision is made to abandon or restore operations.
- The Abandon Option involves ceasing all activity, disposing of all equipment and materials, and abandoning the site.

Three ultimate conditions can result from the Mothball Option:

1) Mothball for 20 years then restore to full operating condition.
2) Mothball for 20 years then permanently abandon the structure.
3) Mothball for 20 years then permanently abandon the structure and demolish the structures (remove to below river bottom elevation).

Two ultimate conditions can result from the Abandon Option:

1) Abandon the structure.
2) Abandon the structure and later demolish the structures (remove to below the river bottom elevation).

U.3 Inventory of Systems

U.3.1 Physical Components

The Lower Granite Lock and Dam main structures include a six-unit powerhouse, an eight-bay spillway with stilling basin, navigation lock, fish facilities, concrete non-overflow sections, and an earthfilled embankment on the north shore.

The following systems must be considered when decommissioning the four lower Snake River dams:

1) Powerhouse systems
   a) Generators
   b) Circuit breakers
   c) Governors
   d) Dewatering pumps and sumps
   e) Transformers
   f) Lubricating systems
   g) Diesel and gasoline tanks
   h) Compressed air systems
   i) Emergency diesel generators
   j) Control room
   k) Main units (hydraulic turbines)
   l) Cranes
   m) Heating and ventilation systems
   n) Elevators
   o) Fish diversion screens
   p) Lighting systems
   q) CO_2 system
   r) Station batteries
   s) Oil storage in the powerhouse
   t) Storage (warehouse, paint, etc.)
   u) Domestic water wells and treatment
   v) Wastewater treatment plant

2) Navigation lock systems
   a) Upstream navigation lock gate
   b) Downstream navigation lock miter gate
   c) Navigation lock drain and fill valves
   d) Navigation lock low level dewatering systems
   e) Navigation lock floating guidewall

3) Spillway systems
   a) Spillway gates
   b) Gate operating machinery

4) Miscellaneous systems
   a) Fishway equipment
   b) Fish entrances
   c) Transportation and collection channel
   d) Attraction water supply
   e) Diffusers
   f) Counting station
   g) Fishway system control
   h) Fish collection system
   i) Visitor and viewing building
   j) Fingerling bypass
   k) Fishway watering and dewatering equipment
   l) Telephone system
   m) Radio base station

n) Mobile cranes
o) Power distribution system (4160 volts, 500 kilovolts, 480 volts, and 120 volts)
p) Juvenile fish hold and load facility

5) Station Service Power requirements
   a) Energy from public utility
   b) Bonneville Power Administration (BPA) Switchyard/Power Co. Construction

U.4 Decommissioning Options

U.4.1 Mothball Option Required Actions

This decommissioning option would require that maintenance be continued on many of the hydropower facilities systems. Maintenance requirements, removal of equipment, or other preparation requirements were evaluated for the mothball option. Table U1 shows all of the significant maintenance or construction activities that would be required for the Lower Granite Dam if the mothball option were selected.

Table U1. Mothball Option Requirements

Feature	Required Maintenance or Construction Activity
Generators and Turbines	1. Place the thrust-bearing pump on timer.
	2. Leave the turbine runner full of oil.
	3. Provide a method to keep turbine oil topped off.
	4. To avoid damage to the thrust bearings, jack up the generator rotor to relieve pressure on the bearings due to the weight of the rotor.
	5. Provide a power source to maintain the present control system to insure the proper ambient temperature.
	6. Provide heating and ventilation systems.
	7. Leave the heaters on in the generator housing.
	8. Put in the head gates and the tailrace stop logs. Some head gates would need to be assembled.
Circuit Breakers	1. Maintain all governors on a regular maintenance schedule.
	2. Exercise the wicket gates and blades each month.
Governors	1. Maintain all governors on a regular maintenance schedule.
	2. Leave governor and oilhead full of oil.
	3. Maintain governor pumps and air compressor.
	4. Exercise the wicket gates and blades each month.
Dewatering Pumps	1. Maintain the dewatering pumps on their current maintenance schedule to avoid flooding the galleries.
	2. Make sure bulkheads are in place during dewatering.
	3. Maintain annunciation systems in operating condition.
Transformers	1. Maintain the nitrogen system on the regular maintenance schedule.
	2. Provide backup power by connecting to the local public utility for station service power.
	3. Check oil for moisture on a regular basis; check insulator status on a regular basis.

Table U1 continued. Mothball Option Requirements

Feature	Required Maintenance or Construction Activity
Lubricating Systems	1. Maintain the lubricating systems on a regular basis. Lubricating systems include hydraulic oil storage tanks, lubricating and transformer oil storage tanks, governor oil, pumps, piping, oil heaters, spillway gates, etc.
Diesel and Gasoline Tanks	Maintain the fuel pump on an annual basis.
Compressed Air Systems	1. Replace the air system lubricants on a regular basis. 2. Prepare the navigation lock air system be fore long-term storage and maintain the 125-psi compressed air system in the powerhouse. 3. Exercise units on a monthly basis.
Emergency Diesel Generators	1. Keep the 500-kilowatt station emergency generator on line to provide power to the alarm system and the drainage pumps, in the event the back-feed power from the BPA line is lost. 2. Replace lubricants on a regular basis and exercise and test-operate each unit on a weekly basis.
Control Room	Keep the sequential event recorders operational for the annunciation system as well as for the phone line for external communications.
Main Units (Hydraulic Turbines)	1. Jack up and block the turbines to relieve pressure on the thrust bearings. Lubricate the bearings and the internal blade components and/or immerse them in oil to avoid corrosion. 2. Paint all surfaces that have a tendency to rust with an appropriate rust preventative paint. 3. Exercise the blades regularly to keep the internal parts freed up.
Cranes	1. Heat the gear boxes and protect the wire rope. 2. Maintain atmospheric conditions within the power house to avoid corrosion on the bridge crane. 3. Remove all wire rope from the cranes that are located outside. 4. Remove the diesel engine from the intake crane and store it within the powerhouse.
Heating and Ventilation Systems	Maintain heating and cooling requirements for the proper operation of the generator. Existing water source heat pumps would be inactive without a water source. A well could be drilled to supply water; however, it would be more economical to replace present heat pumps with electric heat pumps. The local public utility district would provide the power supply for the heat pumps by back-feeding power into the plant.
Elevators	Maintain the annual service contract with outside vendors.
Fish Diversion Screens	Remove and store the screens to reduce corrosion.
Lighting Systems	Replace lamp fixtures, as necessary, to provide a safe working environment.
Station Batteries	Record alarms using an alternating current system with an uninterruptable power system for back up. Remove the batteries.
Domestic Water	Maintain the water system on the current maintenance schedule.
Wastewater Treatment Plant	Maintain all sewage lift pumps and associated equipment on the annual maintenance schedule.

Table U1 continued. Mothball Option Requirements

Feature	Required Maintenance or Construction Activity
Navigation Lock Systems Upstream Gate	1. Leave the gates on the latch pins to take tension off the lifting cables. 2. Maintain the oil- filled gear boxes in the lifting machinery of the upstream tainter gate. 3. Install both the upstream and downstream tainter valve bulkheads. 4. Bulkhead off the gates and drain the lock completely. 5. Block the gates, then close and lock the machinery rooms.
Downstream Miter Gate	1. Dewater the locks. 2. Install the floating bulkhead downstream to dry up the miter gate. 3. Use blocks to seal the floating bulkhead in case the tailwater goes up or down. 4. Leave the 914-millimeter (mm) (36-inch) valve open to the sump to keep the locks drained. 5. Leave the gate in the open position. 6. Provide cathodic protection of the bottom bearing, as necessary, since it will be under water. Change hydraulic oil on a regular basis.
Drain and Fill Valves	1. Change the oil on a regular basis and grease the bearings 2. Install a bulkhead in front of the fill valve in the lock. 3. Operate the drain valves occasionally.
Low Level Dewatering Systems	Lubricate all systems and provide cathodic protection, as necessary.
Floating Guidewall	Move the guidewall into the navigation lock before drawdown and store it in the navigation lock.
Spillway and Gates	Leave the gates closed if they are left in place.
Miscellaneous Systems	Drain air, water, oil, etc. from any of the following miscellaneous systems and leave them in place: fishway equipment, fish entrances, transportation and collection channel, attraction water supply, diffusers, cunting station, fishway system control, fish collection system, visitor and viewing building, fingerling bypass, fishway watering and dewatering equipment, telephone system, radio base station, mobile cranes, power distribution system (4160 volts, 500 kilovolts, 480 volts, and 120 volts), and juvenile fish hold and load facility.

The study team also evaluated the different equipment and hydropower facility features at the other three dams compared to Lower Granite Dam to identify the major differences that would increase the effort (and resulting costs) for decommissioning at those sites. The major differences are as follows:

- Little Goose fish pumps are turbine driven whereas Lower Granite fish pumps are electrically driven. Turbine driven pumps would require additional labor to prepare them for extended outages. Corrosion of the unit would be a major concern.
- The Lower Monumental downstream and upstream navigation lock gates are lift gates, as is the downstream gate at Ice Harbor, whereas the navigation lock gates at Lower Granite are miter gates and tainter gates. Lift gates require the use of cables in their operation, and their storage would need to be considered if long-term outage is expected. In order to prepare the cables for storage, the load on the lifting cables would have to be lessened by the blocking of the counter weights. Once this is

done, the cables could be removed for storage in the power house. This is necessary in order to prevent deterioration of the cables.

- Ice Harbor Lock and Dam has eight fish pumps on the south shore rated at 250 horsepower and three on the north shore rated at 200 horsepower. Lower Granite has two fish pumps at 800 horsepower each.

U.4.2 Abandon Option Required Actions

This decommissioning option would require that only minor maintenance activities be performed to maintain hydropower facility lighting. However, several non-maintenance items would need to be completed prior to and during drawdown. Table U2 shows the required activities if the Abandon Option is selected.

Table U2. Abandon Option Requirements

Feature	Required Activity
Equipment and Property	Excess all equipment and other government property.
Diesel and Gasoline Tanks	Remove the abandoned fuel tanks and disposed of in accordance with environmental requirements.
Lighting Systems	Replace lamp fixtures as necessary to provide a safe environment.
Navigation Lock Upstream Gate	1. Install both the upstream and downstream tainter valve bulkheads. 2. Bulkhead off the gates and drain the lock completely. 3. Block the gates, then close and lock the machinery rooms.
Navigation Lock Downstream Miter Gate	1. Dewater the locks. 2. Install the floating bulkhead downstream to dry up the miter gate. 3. Use blocks to seal the floating bulkhead when the tailwater goes up or down. 4. Leave the 914-mm (36-inch) valve open to the sump to keep the locks drained. 5. Leave the miter gates in the open position. (Close the lift gates.) 6. Provide cathodic protection of the bottom bearing, as necessary, since it would be under water. Change hydraulic oil on a regular basis.
Navigation Lock Floating Guidewall	Store the guidewall in the navigation lock.
Spillway and Gates	Leave the gates closed and remove the wire ropes.

U.4.3 Recommended Option for Implementation

The four lower Snake River hydropower facilities range in age from 23 to 48 years old. It is clear from the maintenance records that the older projects are exhibiting problems associated with aging equipment. Much of the equipment is at the extreme end of its useful life and would likely require replacement during a project restart. It would not be economical to maintain the equipment for 20 years and then have to replace it if the hydropower project is restarted.

Furthermore, the cost of removing and relocating equipment, considering its age, is excessive. Much of the equipment is customized for its current location and would require modification for use by other Federal hydropower facilities. The conclusion of this study is that, as a whole, there is no economic salvage value for the equipment at each of the plants. Consequently, this implementation plan is based on abandoning the hydropower facilities.

The Abandon Option requirements associated with decommissioning would be performed during and after drawdown. The only item that needs to be completed before drawdown is the construction associated with providing power from the public utility. Power for lighting and security would be needed when power production is stopped at the dams.

U.5 Hazardous Materials, Substances, Chemicals, and Wastes

U.5.1 General

Each dam site has numerous items that can be classified as hazardous/dangerous materials, substances, chemicals, or wastes under Federal and state laws. In the event the hydropower facilities are decommissioned, all items that are designated as solid wastes would need to be identified, characterized, and disposed of in accordance with Federal, state, and local regulations and codes.

U.5.2 Inventory of Hazardous Materials, Substances, Chemicals, and Wastes

The following is a list of hazardous materials, substances, chemicals, and wastes normally found at a hydropower facility that may require disposal actions if not recycled or reused for their intended purpose:

1) Polychlorinated Biphenyls (PCBs)
2) Asbestos
3) Paint/abrasive blast grit (red lead paint)
4) Oil
5) Mercury
6) Antifreeze
7) Halogenated and non-halogenated solvents
8) Greases
9) Pesticides (includes herbicides, insecticides, and wood preservatives)
10) Petroleum contaminated
11) Chlorinated fluorocarbons (CFCs) Freon/Halon
12) Gasoline/diesel (includes product and sludge in tanks)
13) Batteries (includes acid)
14) Water treatment sludge (septic tanks/wastewater treatment)

U.5.3 Regulatory Overview and Assumption

Many of the materials and items listed above would meet the definition of a solid waste given in 40 Code of Federal Regulation (CFR) 261.2 and the Washington Administrative Code (WAC) 173-303. This would determine the item's ultimate disposal either as a solid waste or a dangerous/hazardous waste, depending on its condition at the time of decommissioning (e.g., whether it was used material or contaminated material). This study team based its recommendations and costs on the assumption that the potential wastes listed above would require disposal in a Federal- or state-permitted municipal landfill or treatment, storage, and disposal facility (TSD). It should be noted that many of the materials and items listed above (if unadulterated with other regulated hazardous contaminants) could be recycled or used at other Corps hydropower facilities, thereby eliminating the need to dispose of these materials.

U.5.4 Discussion/Recommendations

1) PCBs

   PCBs are still present in small amounts at most facilities, primarily in light ballasts and capacitors. PCB-contaminated articles (e.g., transformers, drums, light ballasts) would have to be properly disposed of if decommissioning occurs. Disposal requirements given in 40 CFR 761 would determine ultimate disposal costs.

2) Asbestos
Most asbestos has been removed from the dam facilities. There are still some locations, such as breaker panels, where removing the asbestos is not feasible or necessary unless it becomes fixable or damaged and presents an exposure hazard to employees. Asbestos disposal requirements given in 40 CFR 61 would determine ultimate disposal costs.

3) Paint
Most external structures at the dams are coated with high concentrations of lead-based paint. If lead-based paint is removed from structures during decommissioning, the resulting paint waste must be tested to determine if it is a toxicity characteristic hazardous waste. Paint waste that exceed the regulatory level for lead or other contaminates (40 CFR 261.24) must be disposed of in a Federal-/state-permitted TSD. Old, unused, and contaminated (mixed with solvents) paint in containers would also have to be disposed of as a solid or hazardous wastes, depending on laboratory analysis results.

4) Waste or used oil
Each hydropower facility has at least 570,000 to 950,000 liters (150,000 to 250,000 gallons) of oil currently in use. This oil would have to be properly disposed of in the event of decommissioning. Oil removed from the turbines and other equipment, including transformer oil, would be either a waste oil (40 CFR 761) or used oil (CFR 279, WAC 173-303-515), depending on prior use and contaminants found in the oil. Containerized oil containing contaminants such as solvents are commonly encountered at hydropower facilities. Oil sludges are common in tanks. Oil disposal would likely be costly due to the large volumes found at hydropower facilities and the ease of contamination with other regulated hazardous wastes.

5) Mercury
Fluorescent light bulbs and switches are the primary sources of mercury waste at these facilities. Thermostats or temperature regulating switches could be disposed of under the Universal Waste Rule (40 CFR 273). Other metallic mercury wastes found at hydropower facilities should be in minimal amounts; however, the disposal of metallic mercury waste not covered by the Universal Waste Rule might require disposal by incineration, which is costly even for small quantities.

6) Antifreeze
Antifreeze must be recycled or disposed of as a regulated dangerous/hazardous waste (WAC 173-303). Most, if not all, used antifreeze generated at hydropower facilities can be reused or recycled. Most municipal landfills accept used, uncontaminated antifreeze. Since commercial vendors who perform routine maintenance on hydropower facility fleet vehicles dispose of the antifreeze from these motor vehicles off-site, only small quantities of anti-freeze waste from cranes, vans, and sedans serviced on-site would need to be recycled or disposed of as a dangerous waste.

7) Solvents
Solvents are used extensively for degreasing operations at hydropower facilities and are probably the second largest source of potential regulated hazardous wastes found there. Solvents are used as thinners for painting applications, and aerosol containers of degreasers and solvents are prevalent in maintenance shops. Spent solvents mixed with used oil is a common source of regulated waste at hydropower facilities.

8) Greases
Greases use on the turbine units and other equipment would have to be disposed of, especially in locations where there is direct contact with the water. In most cases, greases can be burned for energy recovery (WAC 173-303-510). Greases in their original containers can be reused or recycled.

If not contaminated with a regulated chemical or material, greases in open containers can also be reused or recycled.

9) Pesticides
Hydropower facilities use pesticides, (herbicides, and insecticides) on the levees or in and around the facilities for insect and weed control. Unless contaminated with other regulated wastes, pesticides can be reused at other hydropower facilities in accordance with registered label directions. Rinsed pesticide containers can be recycled or disposed of as a solid waste. Unused/waste pesticides can be disposed of at the Washington State Department of Agriculture designated hazardous material collection sites. Disposal of pesticides designated as dangerous waste is regulated by the Washington Department of Ecology (WDOE). Pesticide wastes requiring disposal should be minimal at hydropower facilities if they have been managed correctly.

10) Petroleum-Contaminated Soil
Petroleum-contaminated soils are typically cleaned up as the spills happen, but there could potentially be some areas where the soil has been contaminated early in the history of the facility. Petroleum contaminated soil form past activities may be encountered when soil is removed during the decommissioning of the dams. These areas would have to be studied and analyzed to ensure that no petroleum contaminated soil above WDOE cleanup standards remain. Soil permitted landfill facility, treated on site, or land farmed which reduces disposal costs.

11) CFCs and Halons
Freon and Halon are used as refrigerants/ coolants (heat pumps) , and in fire extinguishing systems respectively at hydropower facilities. These ozone-depleting compounds would have to be properly recycled or reclaimed to comply with Clean Air Act's or quality standards. Additionally, CFCs are also found in used oil after reclamation and therefore may be subject to the used oil disposal requirements mentioned above. Special requirements of CFCs are given in WAC 173-303-506.

12) Gasoline and Diesel
Gasoline and diesel storage tanks are located a each facility. These tank must be emptied of product in the event of decommissioning. Unused product can be used for fuel. Gasoline and diesel contaminated with water can be burned for energy recovery. Sludge disposal costs must be considered when gasoline and diesel tanks are emptied. Sludges remaining in both the diesel and gasoline tanks may contain regulated waste compounds (benzene) or metals (lead) and may be disposed of as dangerous wastes.

13) Batteries
Batteries, including spent lead-acid batteries, and battery acid (electrolyte), used in power houses and other facilities must be disposed of if decommissioning occurs. Battery disposal requirements are given in 40 CFR 273, and WAC 173-303. Spent batteries and electrolytes must be reclaimed or disposed of as a regulated hazardous/dangerous wastes.

14) Wastewater Treatment Sludge
Powerhouse wastewater treatment and septic tank sludge must be removed and disposed of if decommissioning occurs. Federal requirements for treatment and disposal of sewage sludge are given in 40 CFR 403 and 503. Sewage sludge (biosolids) may contain toxic pollutants (metals) which impact disposal options. Sewage sludge are generally excluded from Federal and state hazardous/dangerous waste disposal requirements (40 CFR 261.4 and WAC 173-303.071). Sludge must be tested for toxic pollutants prior to disposal.

U.5.5 Disposal Sites

Specific disposal sites for the listed hazardous materials were not specifically identified. Several commercial sites are currently in operation in the Pacific Northwest. Those facilities are able to handle the quantity of listed materials that would eventually be transported to such a site. Note that many of the listed materials are products that are useful to power facilities on the Columbia River. Disposal requirements make those materials available for use by others before disposal is considered.

U.6 Hydropower Facilities Security

U.6.1 General

Currently, the public has access to most of the exterior of the hydropower facilities, including the tailrace, forebay, fish facility, navigational lock, and the top deck of the dam. All four projects provide a public roadway across the river generally available for public use during daylight hours. All entrances to the interior of the hydropower facilities have secured access requiring authorization and a key. There is little restriction on public water access to the navigation lock for upriver or downriver lockages, however, access to the hydropower facilities, the approach channels, and the discharge and spillway channels are restricted.

There are three possible options for security at the abandoned hydropower facilities after drawdown:

- Option A. Use and expand the current security system.
  The current security system consists of closed circuit video monitoring and keyed locks on all access doors to the interior of the hydropower facilities. This system could be upgraded and tied into the current radio system for alarm purposes to the local authorities.

- Option B. Contract out to a private concern.
  Contracting with a 24-hour-monitoring service and upgrading any controllers necessary would allow for the addition of fire and environmental monitoring. This type of monitoring also could be done remotely.

- Option C. Install security fences and signs.
  Access to the hydropower facilities by vehicular traffic would be denied by blocking all roads with chain link fences at each entrance to the facility. Government property signs would be installed indicating no trespassing on government property. Fencing would also have to be installed on the perimeter of the structure to deny access to anyone entering either by foot or by boat, either up or downstream. All access doors to the structure would be locked and all windows screened. This option would also include periodic, manned surveillance of the hydropower facilities.

U.6.2 Recommended Option

In the event that drawdown does occur and the hydropower facilities are completely shut down, the study team recommends that fencing and signs be installed and manned periodic surveillance be performed as stated above in Option C.

Annex V

Concrete Structures Removal Plan

Annex V: Concrete Structures Removal Plan

V.1 General

The fundamental method of establishing a natural river is to remove the embankments at each of the four lower Snake River dams and leave the concrete structures in place. This annex presents the additional steps needed to fully remove visible concrete structures. This is a concept-level investigation since it considers only gross quantities of materials in the major structures.

V.2 Description of Existing Structures

Each hydropower facility on the lower Snake River contains at least four major concrete structures that would be removed: the powerhouse, navigation lock, spillway, and fish facilities (comprised of fish ladders and juvenile fish facilities). Several smaller structures such as non- overflow sections of dam, offices, visitors center, parking lots, roads, and bridges would also be removed. Non-concrete structures include embankment sections not otherwise removed for the new channel and numerous steel structures on the project.

In addition, all mechanical and electrical equipment would be removed, including major generating equipment, (turbines, generators, governors, and exciters) auxiliary equipment, head gates, bulkheads, spillway gates, trashracks, fish screens, transformers, switchyards, and transmission substations.

Following is a brief description of the major structures:

- Powerhouse
  Each site has a six-unit powerhouse with adjacent erection bay. The structure is approximately 200 meters (m) (656 feet) long by 75 m (246 feet) wide. The upstream wall of the powerhouse is essentially a concrete dam that serves as the intake.

- Spillway
  Each site has an eight-bay spillway with radial gates, except for Ice Harbor Dam that has 10 bays. Each bay is 15.2 m (50 feet) wide, separated by piers that are 3.05 m (10 feet) wide at Ice Harbor and 4.3 m (14 feet) wide at the other three sites. Little Goose Dam has a flip bucket, and the other three sites have a stilling basin. The stilling basins would be below average riverbed and, therefore, would be left in place.

- Navigation Lock
  Each site has a navigation lock with an inside length of approximately 245 m (804 feet). Walls are massive concrete approximately 47 m (154 feet) high with a top width of 12 m (39 feet) to 15.2 m (50 feet).

- Fish Ladder
  Each project has a fish ladder with three entrances: one each on the south shore and north shore, and one end of the powerhouse. Fish ladders are concrete flumes approximately 360 m long (1,181 feet) and supported on concrete columns. Each ladder has a pump system to supply extra attraction water.

- Juvenile Fish Facilities
  Each project has a juvenile fish facility consisting of piping, metal flumes, concrete raceways, a laboratory building, and, in some cases, a separator holding and loading facilities.

- Non-Overflow Dams
  Each project has non-overflow concrete gravity dams that join the main structures or serve as cut-off walls. These dams are up to 45 m (148 feet) high and have a back slope of 3 on 4.

- Miscellaneous Buildings
  Each project has some miscellaneous facilities, such as visitors center, offices, parking, roads, and bridges.

- Embankment Dams
  Dams on the shore opposite the first stage channel would be left in place above high water level.

V.3 Completed Project Description

If this option were implemented, all structures would be removed from above the natural riverbed elevation, allowing the river to flow in as nearly an original configuration as possible. A plan of the completed project for each of the four sites is shown in Figures V1 through V4. These figures illustrate the concrete rubble storage area and the effect of channel widening performed during river channelization.

The work of concrete removal would begin after the work for embankment excavation and river channelization had been completed.

V.4 Cofferdams

A cofferdam is necessary to isolate the demolition area from the river channel. Demolition must follow the drawdown activities that restore the river to a "natural" condition. Demolition in advance of drawdown would disrupt ongoing operations and impede the actions necessary to implement the drawdown. In the drawdown plan, following embankment removal, a series of channelization levees would be constructed to form the permanent channel around the remaining concrete structures. Those levees are permeable since they are constructed of shotrock, which allows water to freely pass through the levee. This same channelization of the river would be necessary during demolition. However, the levees must be impervious for demolition so that the interior zone can be dewatered before the work can proceed. Since the cofferdams would be temporary structures, they could be constructed of local gravels instead of the pervious shotrock.

These cofferdams, made of local gravel fill, would be similar in design to the cofferdams used for initial construction of the dams. According to contract documents for original construction, a cut-off trench was made through the cofferdam section, down to rock along the centerline, and held open with a bentonite slurry mix. This same process is proposed for construction of these temporary cofferdams. The trench would be constructed after the cofferdam had reached full height.

Instead of simply extending the new cofferdams to the existing concrete structures like the levee, the cofferdams would actually enclose all the concrete structures and rejoin with the shore.

Cofferdams at Lower Granite and Little Goose would involve only one stage as shown in Figures V5 and V6. The new channels at both sites would be sufficiently large to accommodate a cofferdam and still have acceptable velocities for fish migration. Cofferdams at Lower Monumental and Ice Harbor dams would involve two stages as shown in Figures V7 and V8. Two stages are necessary at these two sites because the temporary channels are too narrow to accommodate an earthfill cross section and maintain acceptable velocities for fish migration. The first stage would be a dike system that joins the end of the navigation lock to the shore enclosing the powerhouse and spillway. Behind this cofferdam, removal of

the powerhouse and spillway would be accomplished. The second stage would include removal of the first cofferdam and construction of a second cofferdam enclosing the navigation lock to the opposite and nearest shore.

While the cofferdams and slurry trench were being installed, all equipment would be removed from the site and staged in an area for dismantling and disposal.

Dewatering facilities consisting of multiple pumps and collector ditches would be provided for the interior of the cofferdam areas. Water would be treated locally and returned to the river.

Demolition of concrete structures would be accomplished to an elevation 2 m (6.6 feet) below the average river bottom level. The draft tube passages, approach, and tail channels would be filled-in with concrete rubble. All concrete remaining in the river below grade would be covered with riprap and river bed material to match the average river grade.

Concrete rubble from demolition would be disposed of by placing it in riprap fashion along the bank of the river within the cofferdam area. Horizontal thickness of the concrete rubble would be approximately 30 m (approximately 95 feet). Exposed reinforcing bars and embedded metal would be removed.

After equipment and concrete structures have been demolished, hydraulic excavators working from the cofferdam crest at a point furthest from the shoreline would remove cofferdams. The two equipment spreads would first breach the cofferdam, then excavate in opposite directions towards the riverbank, loading haul trucks to remove the cofferdam material for disposal away from the river edge.

V.5 Concrete Removal (Demolition)

The study team assumed drill and blast would remove all concrete. The team estimated gross quantities and based removal costs on unit prices for drill and blasting mass concrete and reinforced concrete. The team determined that, while other methods of demolition such as hydraulic crushing might be more appropriate for specific structures, such a detailed approach to demolition was not within the scope of this effort. The plan assumed mass concrete would be removed in approximately 6-m (20-foot) lifts by drill and blast.

The team determined that demolition of the powerhouse, spillway, fish facilities, and navigation locks (at Lower Granite and Little Goose) could proceed simultaneously. All holes would be drilled for nominal lift heights of about 6 vertical meters, and the site cleared and blasted. Load and haul to the spoil area could proceed while the next lift of holes were being readied for blast.

The following equipment would be used for demolition:

- Drills - Rotary, air track rigs drilling 5-centimeter (cm) (2-inch) holes would be used.

- Loaders - Maximum size CAT 992D rubber tire loader with 10.7 cubic meter (m^3) (14 cubic yards [cy]) bucket and other sizes (CAT 988F with 6 m^3 (7.8 cy) bucket and CAT950E with 2.5 m^3 (3.3 cy) bucket) would be used to load haul trucks.

- Trucks - Off-road haul trucks (CAT 769C with 19 m^3 (25 cy) capacity and CAT 777C with 46 m^3 (60 cy) capacity) would used, depending on loader capacity.

- Dozers - CATD7H and CATD9N track dozers would be used to push and spread material during excavation and spoiling operations.

- Excavators - CAT245D hydraulic excavators with 2.3 m^3 (3 cy) buckets were selected for general purpose and concrete rubble excavation and riprap placement. CAT 5130 hydraulic excavators with 9.9 m^3 (13 cy) buckets were selected for mass excavation and removal of cofferdams.

- Clamshell - An American 12220 crane with 30.5 m (100 feet) boom and 3 m^3 (4 cy) clamshell bucket was selected to place and remove riprap in water and to remove other materials deeper than 10 m (33 feet) underwater.

Quantities for this study were obtained from the quantity summary tables in the existing project Design Memoranda, as follows:

- Cofferdams were assumed to have the same cross section as was used for the levees of the embankment excavation study.

- Dewatering facilities were based on what was provided during original construction.

- Quantities for concrete structures were based on data from the Design Memoranda.

- Quantities for miscellaneous buildings were included with the powerhouse.

- Maintenance shop and public service facilities were considered incidental structures that were calculated on a square meter basis.

- Parking lots and nearby roads were calculated on a square meter basis.

- Drilling and blasting estimates were based on a typical pattern for blast production rate.

- The volume of rubble was estimated to be 40 percent greater than bulked concrete for each site.

- Quantities for material hauled to the waste area were based on typical load/haul rates with distances of less than a kilometer.

- The excavation rate for cofferdam removal was based on the previously developed excavation rates for embankment removal in Annex B, Embankment Excavation Plan.

- All steel and metal was assumed to be scrap and a nominal salvage value was included.

- The spillway at Ice harbor has 10 bays vs. 8 at the other dams. However, the ogee height is 15 feet less and the pier width 4 feet less than at the other three dams; therefore, the volume of spillway concrete removed at Ice Harbor is, in fact, comparable to the other dams.

- The navigation lock at Lower Granite has approximately 50 percent more concrete than the other three projects.

- The spillway at Little Goose has approximately 50 percent more concrete than the other three projects.

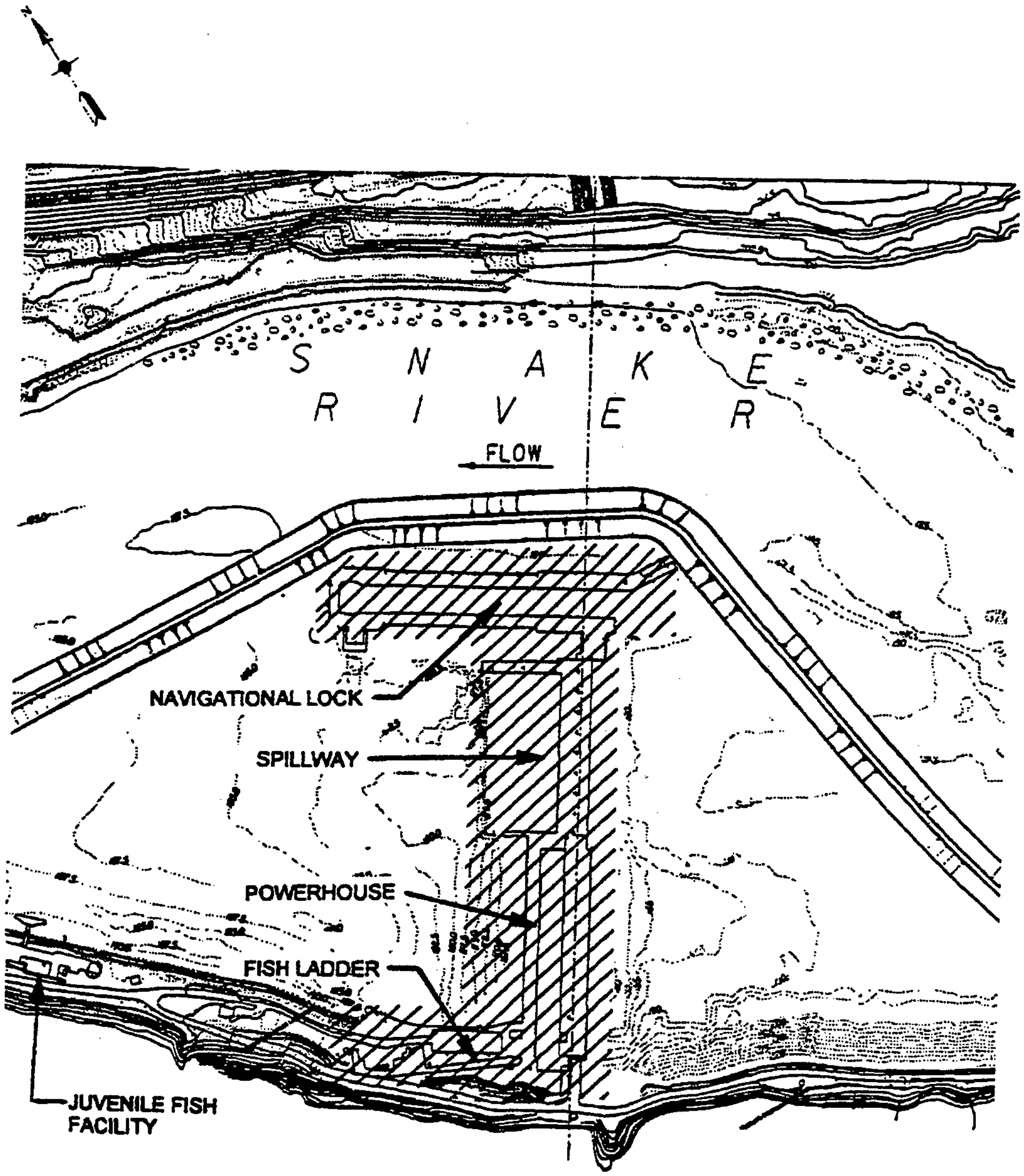
- The non-overflow structures at Ice Harbor have more than twice the volume as any of the other three dams.

V.6 Construction Schedule

The removal of concrete structures would begin soon after the drawdown of the reservoir. Once the reservoir is eliminated, the equipment for producing power is no longer needed.

Removal would begin with the major equipment (turbines, generators, transformers, switchyard equip, gates, screens, and trashracks). Generators would be removed first. Turbine removal could begin as soon as the first generator had been removed. Auxiliary equipment could be removed concurrent with the generators followed by miscellaneous equipment: lighting, cables, control room, fire protection batteries.

At the same time, the temporary cofferdam would be constructed. These cofferdams included an impervious core to facilitate dewatering of the interior areas where concrete demolition is to occur. Concrete removal could begin concurrently after breaching of the embankment dam. Demolition would proceed generally from the top down, piers and non-mass concrete first then the mass concrete and substructures. Crews would probably drill most of the day and clear out for one shoot. The next day crews would come back clean up and proceed with drilling while the previous days shot rubble was being loaded and hauled to the disposal area along the shore. Demolition of one or more major structures could proceed simultaneously.



PLAN
LOWER GRANITE

STEP 1. CONSTRUCT COFFERDAM/LEVEE AROUND ALL CONCRETE STRUCTURES.

STEP 2. REMOVE ALL CONCRETE STRUCTURES.

STEP 3. DISPOSE OF CONCRETE ALONG SHORE LINE WITHIN THE LEVEE.

STEP 4. REMOVE COFFERDAM.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
SEQUENCE OF CONCRETE REMOVAL AND COFFERDAM
LOWER GRANITE - PLAN

Figure: V1

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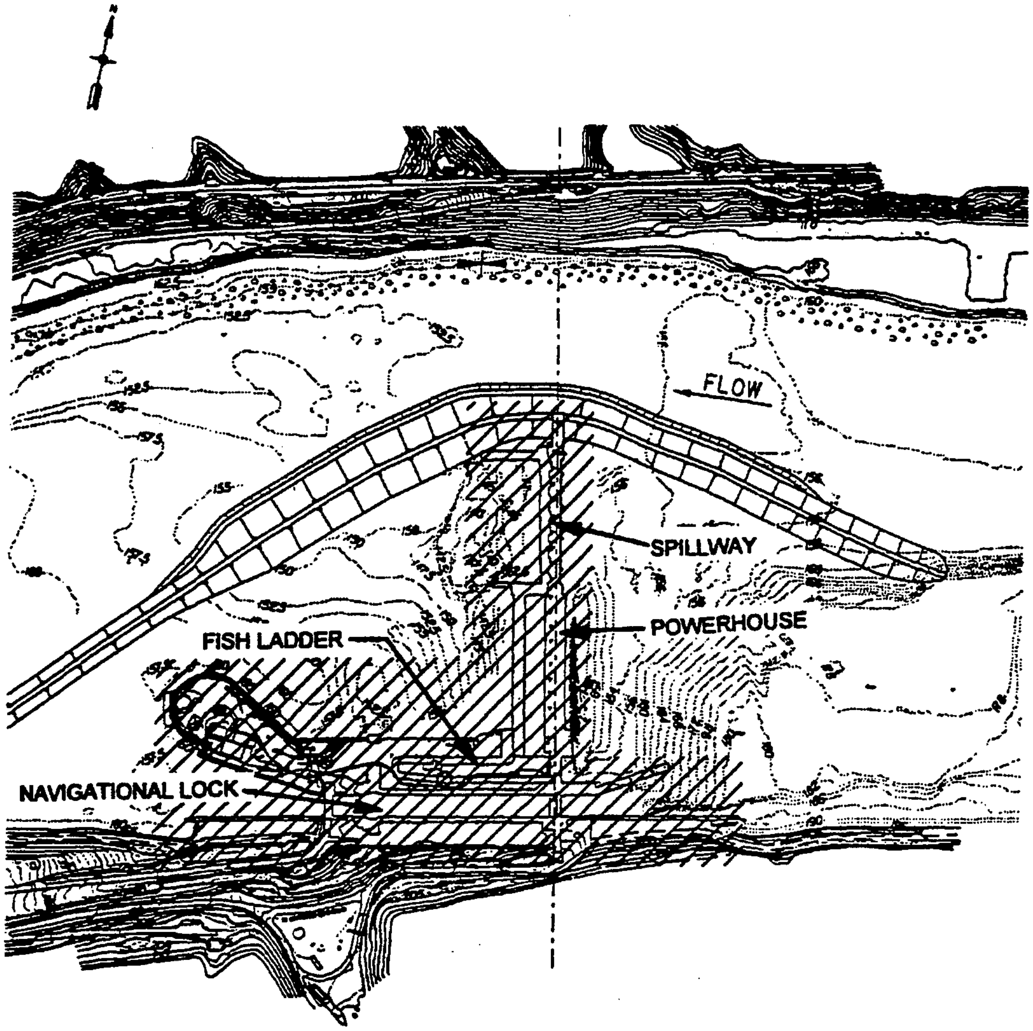

PLAN
LITTLE GOOSE

STEP 1. CONSTRUCT COFFERDAM/LEVEE AROUND ALL CONCRETE STRUCTURES.

STEP 2. REMOVE ALL CONCRETE STRUCTURES.

STEP 3. DISPOSE OF CONCRETE ALONG SHORE LINE WITHIN THE LEVEE.

STEP 4. REMOVE COFFERDAM.

SHEET MAIN SCALE:

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
SEQUENCE OF CONCRETE REMOVAL AND COFFERDAM
LITTLE GOOSE - PLAN

Figure: V2

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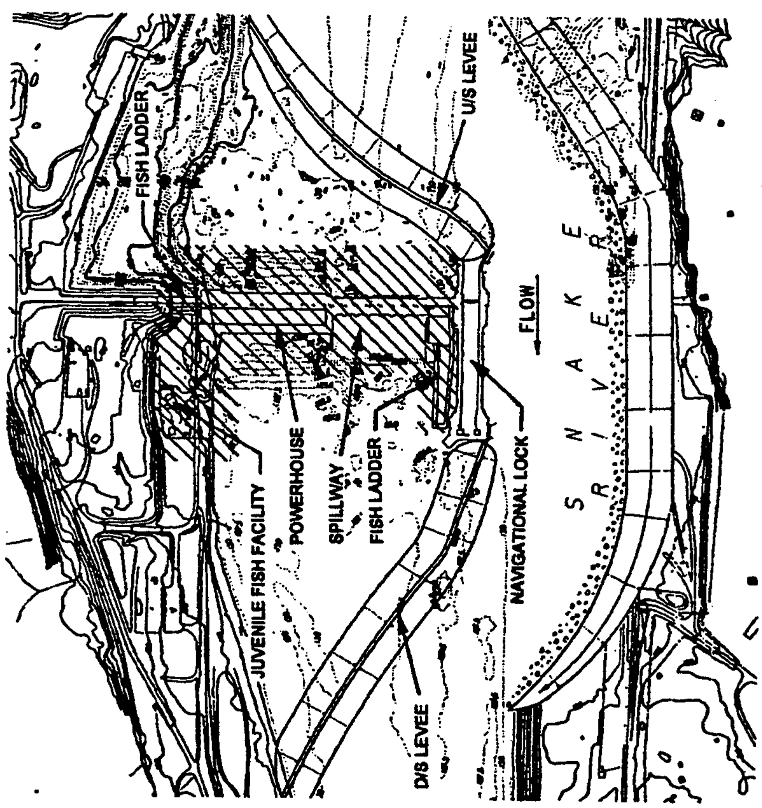


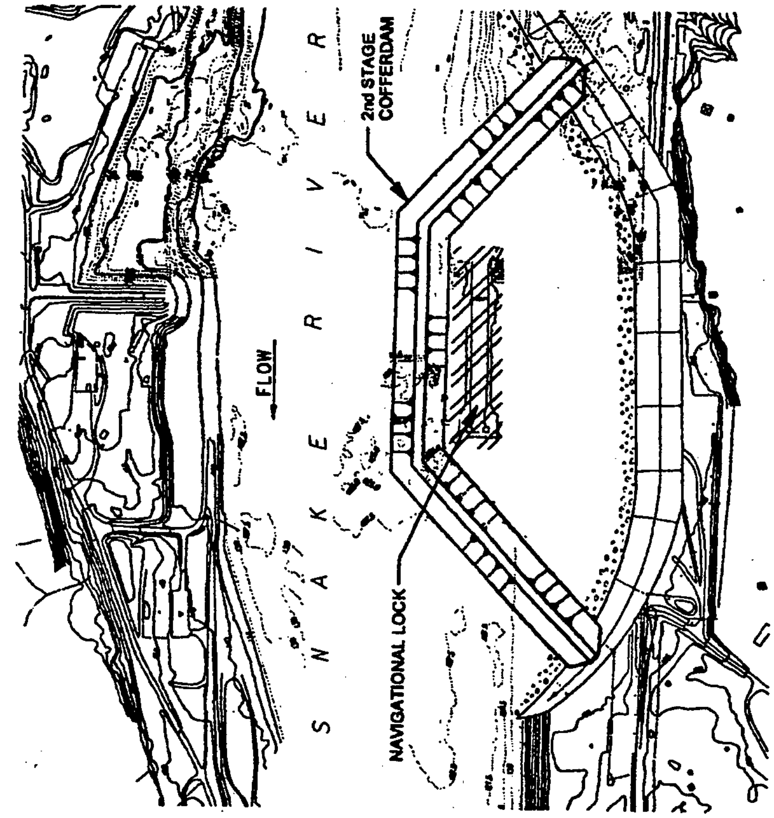
PLAN
LOWER MONUMENTAL

STAGE I

STEP 1. CONSTRUCT COFFERDAM/LEVEE.

STEP 2. REMOVE CONCRETE STRUCTURES (HATCHED AREA) EXCEPT NAVIGATIONAL LOCK

STEP 3. DISPOSE OF CONCRETE ALONG SHORE LINE WITHIN THE LEVEE.



PLAN
LOWER MONUMENTAL

STAGE II

STEP 1. REMOVE LEVEE.

STEP 2. CONSTRUCT COFFERDAM AROUND NAVIGATIONAL LOCK FROM 1st STAGE LEVEE MATERIAL.

STEP 3. REMOVE NAVIGATIONAL LOCK

STEP 4. REMOVE COFFERDAM.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
SEQUENCE OF CONSTRUCTION PHASE I AND 2
LOWER MONUMENTAL - PLAN

Figure: V3

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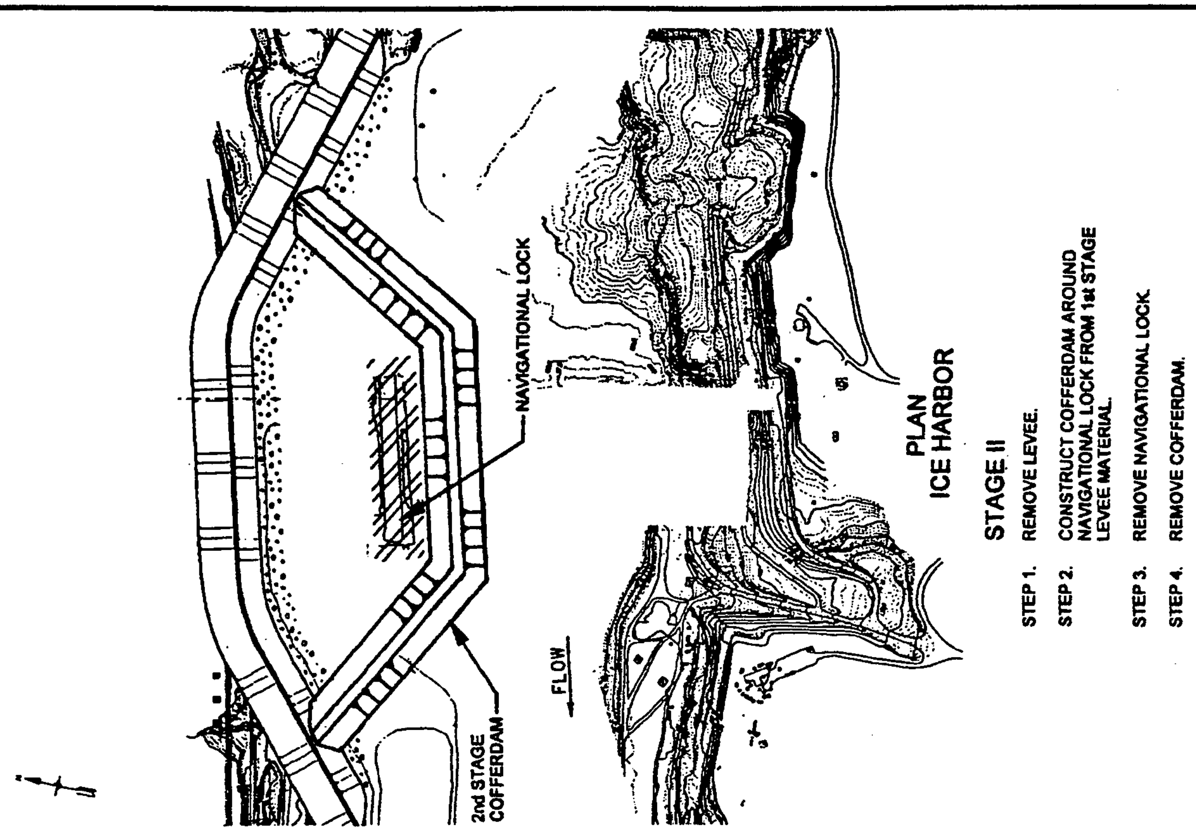
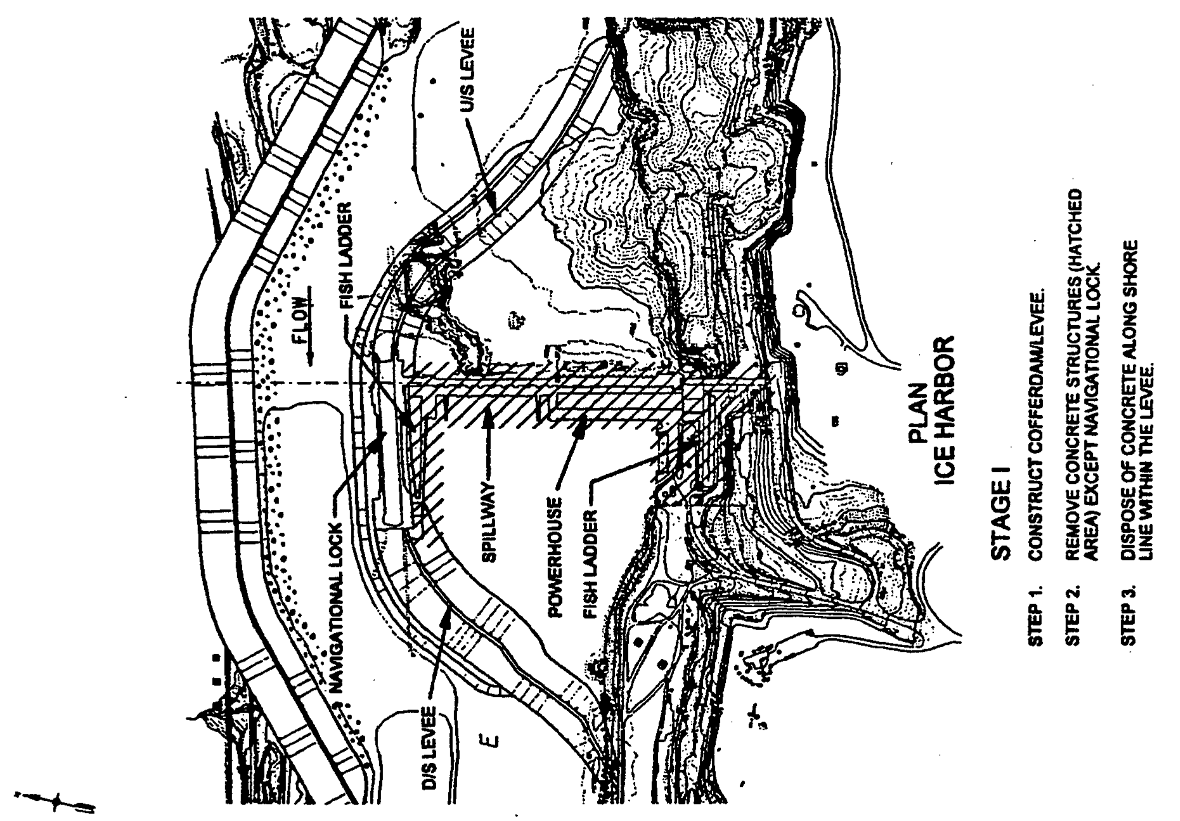
SHEET MAIN SCALE





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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
SEQUENCE OF CONSTRUCTION PHASE I AND 2
ICE HARBOR - PLAN

Figure:
V4

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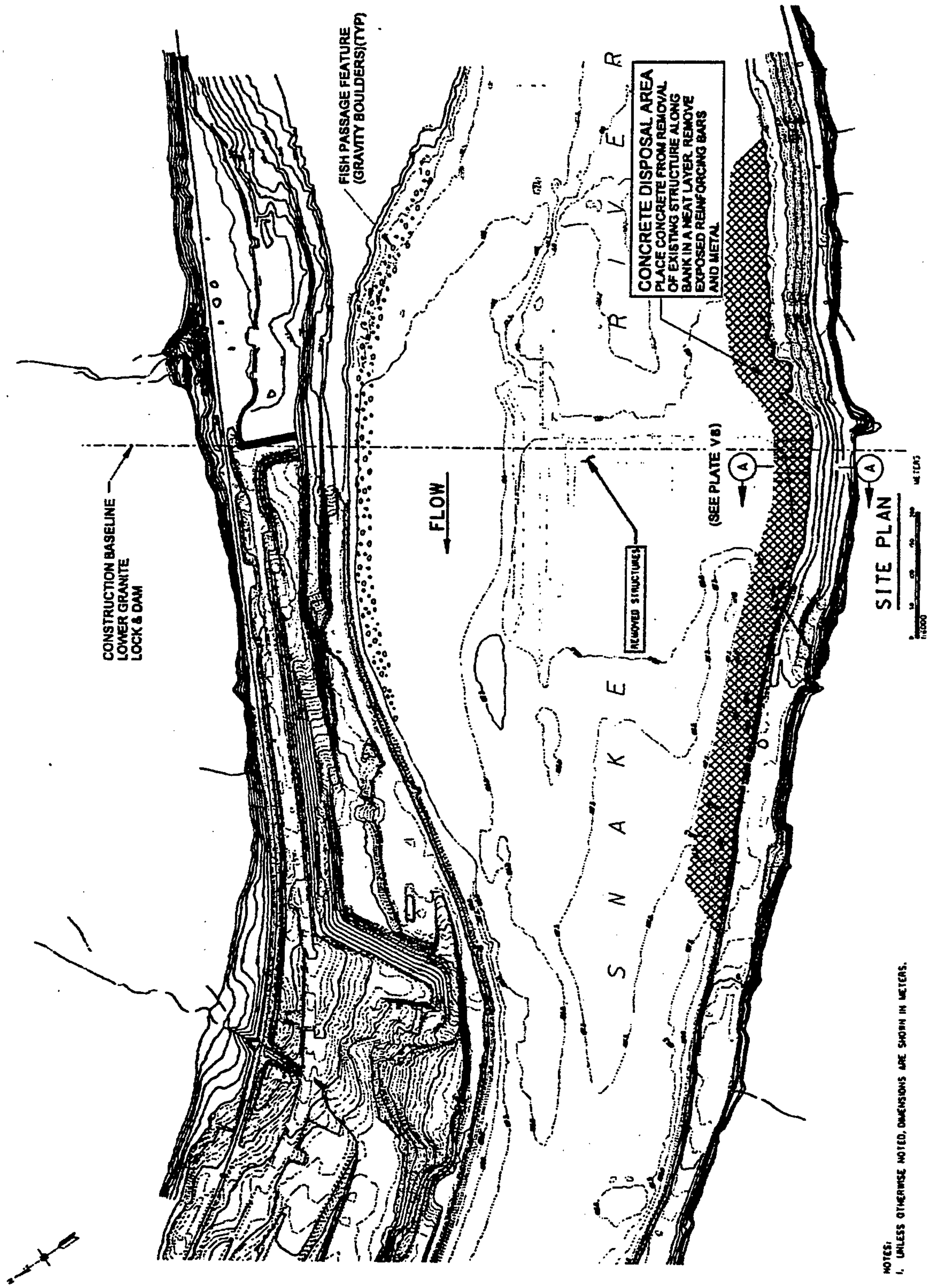

NOTES:
1. UNLESS OTHERWISE NOTED, DIMENSIONS ARE SHOWN IN METERS.

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
LOWER GRANITE REMOVAL OF CONCRETE STUCTURES
COMPLETED PROJECT PLAN

Figure:
V5

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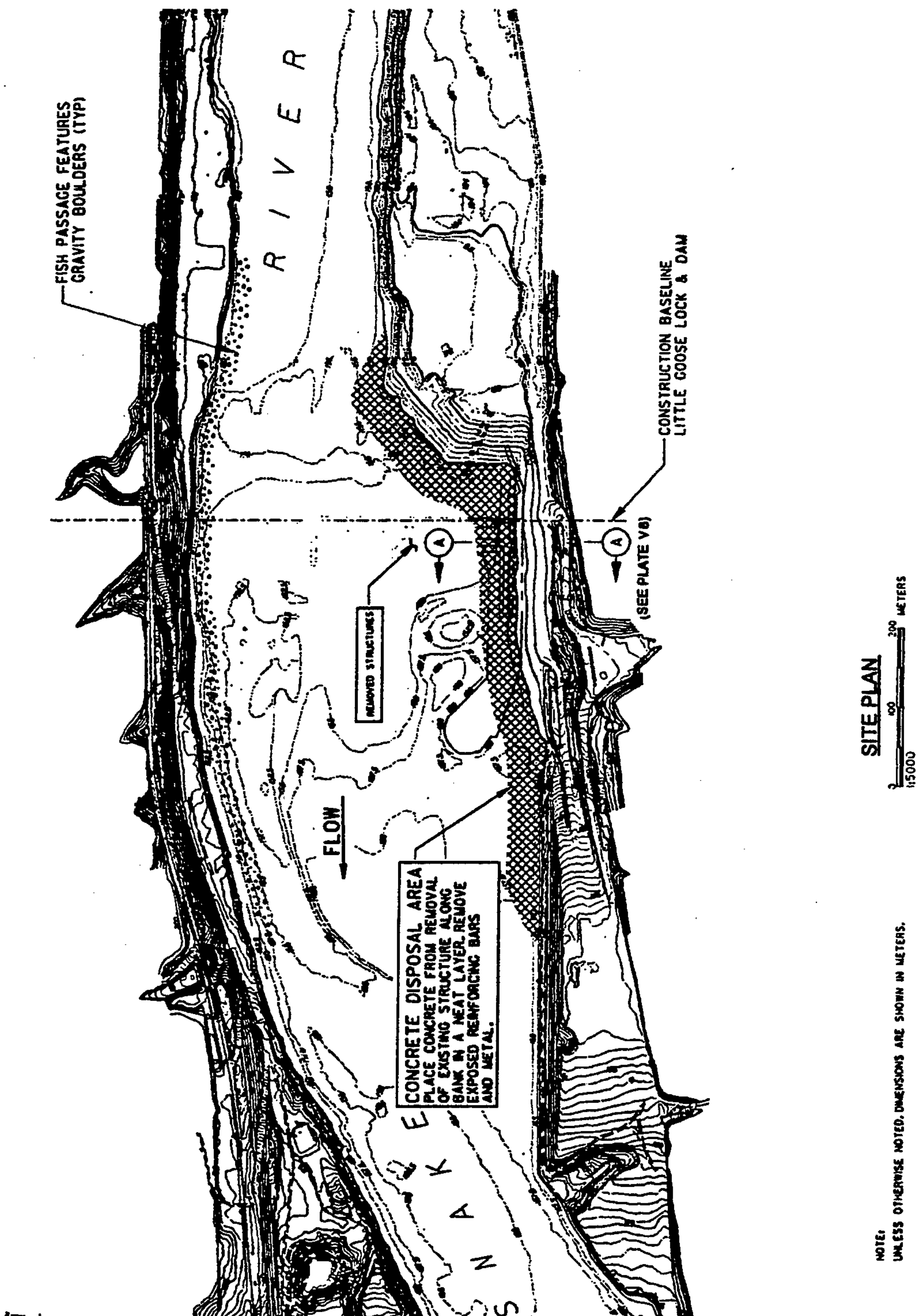

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
LITTLE GOOSE REMOVAL OF CONCRETE STUCTURES
COMPLETED PROJECT PLAN

Figure: V6

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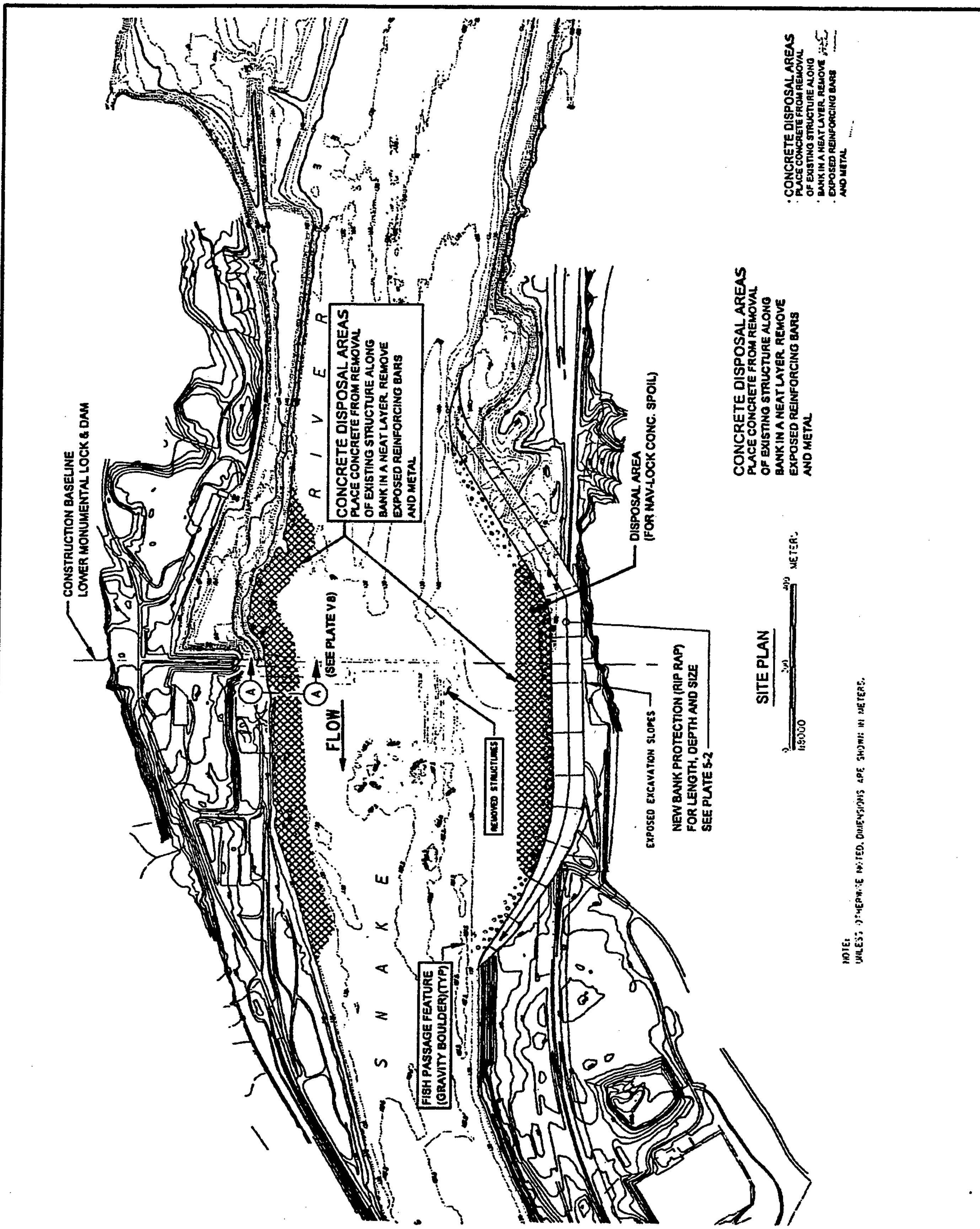

SHEET MAIN SCALE:

US Army Corps of Engineers
Walla Walla District

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
LOWER MONUMENTAL REMOVAL OF CONCRETE STUCTURES
COMPLETED PROJECT PLAN

Figure:
V7

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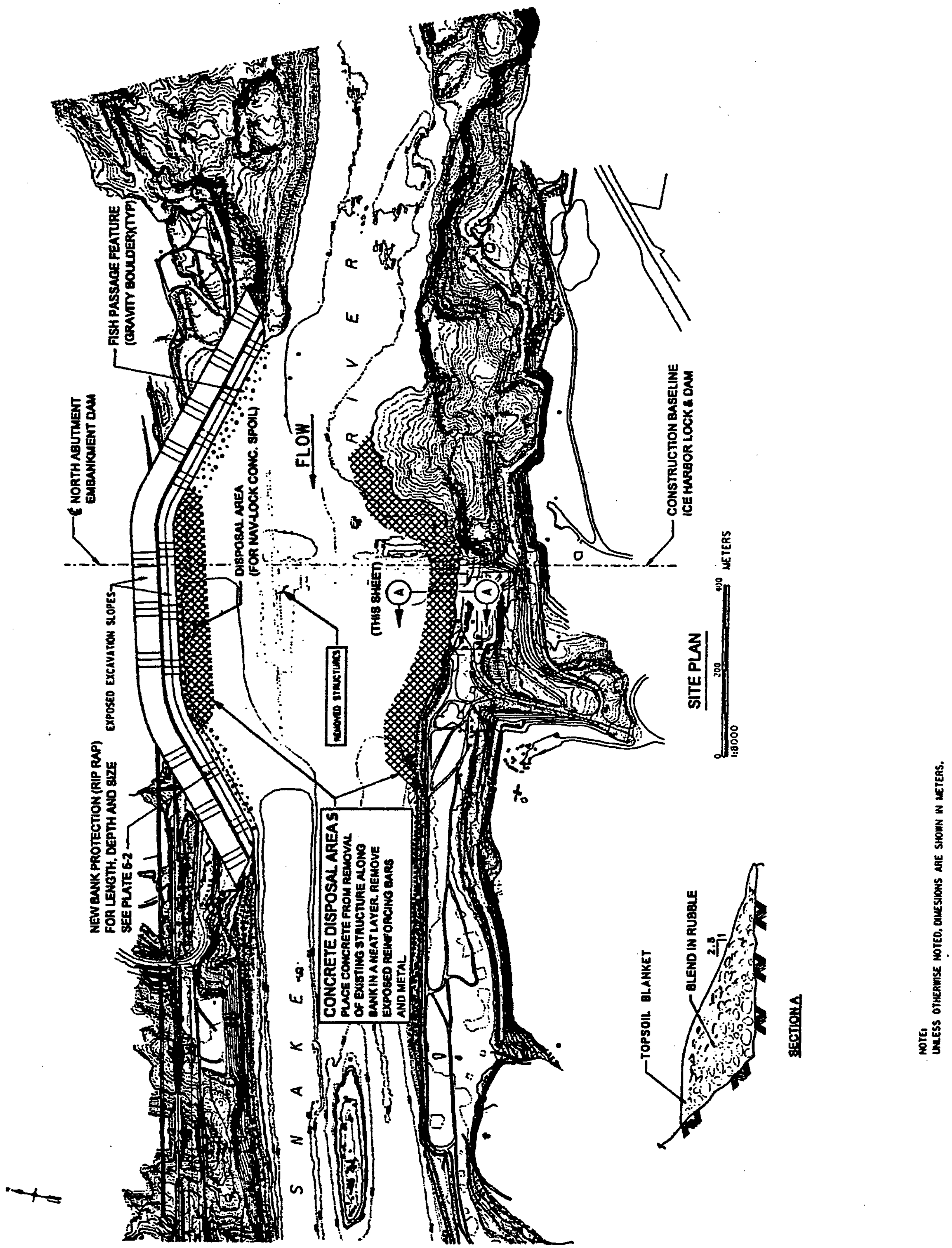

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LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY
ICE HARBOR REMOVAL OF CONCRETE STUCTURES
COMPLETED PROJECT PLAN

Figure:
V8

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Annex W

Implementation Schedule

Annex W: Implementation Schedule

W.1 General

The schedule information presented in this annex was based on the scope of work, assumptions, and methodology presented in the companion engineering annexes (Annexes A through V of this appendix). The following sections summarize the overall project implementation schedule and provide specific details concerning the schedule of the proposed work items.

The general process for implementing the work is to perform a three-step process consisting of 1) preparation of a detailed design report, 2) preparation of contract documents, and 3) performance of construction.

The detailed design report, formerly designated a General Design Memorandum or a Feature Design Memorandum, details the process of identifying, evaluating, and selecting a design option. The activities often are precluded by a survey of each construction site to establish the land configuration. Subsurface explorations using intrusive methods such as drilling, excavating, and sampling and/or geophysical methods such a pulse-velocity, radar, or other subsurface logging methods are conducted at this stage. For some features, hydraulic models must be constructed and flow conditions evaluated for a range of flow and physical conditions. Options are developed for the feature and detailed evaluations are made to select the most favorable option. The selected option is often further developed so that a reliable schedule and cost estimate may be generated.

After review and approval of the detailed design report, preparation of the plans and specifications can proceed. This phase requires completion of the feature design and the development of contract documents. The documents must be prepared in a manner that allows bidders to prepare a realistic bid proposal, that presents features in manner that is constructable, and that provides implementation and operations that address the relevant environmental concerns.

Once a contract has been awarded, the construction can begin. The short-term nature of many of the tasks coupled with the complexity of implementation will require the participation of many individuals and organizations. Construction activity spans a time period of approximately 8-9 years. During the peak years, expenditures are estimated at 200 million in a single year. The bulk of the work is done during a 3-month period. Extensive contractor participation is necessary for this level of effort. Significant administration and construction management participation is also required.

The schedules below reflect reasonable time durations to perform these efforts. They identify time for producing detailed design reports, contract documents, peer and policy reviews, advertising periods and construction operations.

W.2 Overall Implementation Schedule

The implementation of drawdown can be grouped into 3 distinct phases. The preparatory phase is the work necessary to be done in advance of drawdown in order to be able to perform drawdown, to continue operations during drawdown. The drawdown phase is the work required during and immediately following drawdown of the reservoirs. Numerous tasks are anticipated to be performed following drawdown. The period of time that all these occur is shown in Figure W1.

A key decision in implementing drawdown is the sequence of dam breaching. Many options are conceivable. They range from concurrent breaching of all 4 dams in a single construction season to individual breaching of each dam during different seasons with many combinations between.

Breaching individual dams on different years greatly simplifies construction operations and focuses attention on one project at a time. The first project provides a troubleshooting opportunity so that subsequent projects can be breached more effectively. Events that may lead to delays that prevent breaching during the designated season are more effectively controlled increasing the likelihood of on-schedule completion. Funding is less difficult to secure because annual requirements can be spread out over a longer period of time.

Breaching of an embankment structure will generate the migration of embankment silts and sands down river. A much more significant effect is the migration of silt deposits and higher velocity river flows erode those deposits. Silts suspended in the water may be at very high concentrations during the drawdown period of August to December and possibly higher levels during the high flow months of January through June. The effect of this silt and sediment is expected to have a serious negative effect on adult fish migration and a lesser effect on juvenile migration.

If the four dams are breached simultaneously, then this condition will be concentrated to the shortest time period thereby minimizing the negative effects on migrating fish. Biologists expect that expanding this situation as long a four consecutive years could be detrimental to the species (Jones, 1999). Breaching the four dams over two consecutive years provides for realistic implementation of all the construction activity for a time period less devastating than other options that include longer periods.

An aggressive schedule to simultaneously breach 4 dams needs much more detailed evaluation. An evaluation of risks and impacts of specific construction activities is necessary to produce a plan that contains the appropriate backup plans and contingencies to guarantee that the work can be completed in the short timeframe. At the current level of study, it is clear that too many things can go wrong that may force the project into a 2-year breach schedule. Until those uncertainties can be resolved, a 1-year breach schedule cannot be considered.

There is appropriate equipment available to accomplish simultaneous removal of all four dams using one or more contractors. The fewer contractors used, the less the overall cost would be. Each additional contractor used would add approximately 10 percent to the total cost of the individual dam's work. There are three scenarios for removal of the embankments at the four lower Snake River dams:

1. Remove one embankment each year.
2. Remove one or two embankments the first year, gaining experience from that operation, then remove the remaining two or three embankments the following year.
3. Remove all four dams concurrently in one year.

There are significant advantages to removing all four embankment dams in one year. This option would return the river to its natural state for fish migration much sooner. It would also shorten construction duration because reservoirs could be drawn down and some of the work could be accomplished in the dry. The headwater of one dam would be the tailwater of the next dam upstream, and if that reservoir had been drawn down, then construction at the upstream dam could more easily be accomplished either in the dry, or in a lower, quieter flow condition.

W.3 Schedules for Individual Tasks

The following discussion provides explanation of some of the assumptions that support the schedules for the individual tasks.

W.3.1 Turbine Passage Modification

The design of required elements of turbine modifications may result in 2-8 contracts. It is very likely that all the intake gate modifications will be designed and contracted as a group. Likewise work for tailrace draft tube bulkheads, cooling water modifications, and instrumentation systems will be packaged into separate design and construction packages. Groups refer to all the items to be done at one time, e.g. Little Goose and Lower Granite intake gates in one contract.

Sufficient lead time is necessary to order specific parts. Precast construction of tailrace bulkheads requires significant lead time. Most of the work can be completed at any time in advance of drawdown. Target deadlines have been assume to be 60 days in advance of the start of drawdown for the respective project.

The critical element in this feature is the removal of the turbine blades for 3 units at each project. Very early removal of the blades results is a longer period of lost power production. More importantly, turbines must remain operational through the previous spill season in order to minimize spillway usage and limit the consequent gas levels in the river. For this reason some further development of blade removal activity should be considered.

W.3.2 Dam Embankment Excavation and River Channelization

It is anticipated that the contract for breaching and removing the embankment dam will be for both dams scheduled for that season. This work involves developing stockpile or waste areas, haul roads, excavation, bank protection, and restoration of the site. The contract will also include construction of the channelization levees, installation of the permanent fish passage features, and certain elements of the decommissioning of the remaining powerplant, spillway, navigation lock, and appurtenant facilities.

Contract award should provide up to 6 months of lead time contract work in advance of drawdown. It is critical that the breach and embankment removal be completed in advance of 31 December. River channel efforts can be more easily accommodate short periods of high flows during the months of January and February.

The study team believed construction excavation rates could exceed the values assumed for this schedule. However, even with the excavation rates assumed, the drawdown rate of 0.6 m (2 feet) per day governs the length of construction. Beginning drawdown several weeks before the start of excavation and allowing faster excavation rates would shorten the length of time the embankment is exposed to overtopping during construction, but would not reduce the overall length of the in-water construction period, which is governed by the drawdown rate.

W.3.3 Temporary Fish Passage

The major tasks in this feature are modification of the existing fish ladder to incorporate a fish trap and loading facility, the modification of existing fish transport trailers, and the fabrication of new fish transport trailers.

Significant lead time is necessary to fabricate and modify fish transport trailers. Existing trailers must be used to transport juveniles downstream during the spring-early summer migration period. Consequently modifications must allow trailers the ability to switch between juvenile and adult haul mode without major effort. Adult hauling will commence almost immediately following juvenile hauling.

W.3.4 Bridge Pier Protection

Protection measures for bridge piers include the transportation of riprap for bank protection, the installation of sheetpile to encapsulate bridge piers, and the placement of rock and concrete inside the sheetpile enclosures. The in-water work is configured to be done during the in-water work window, although some provision for early extensions is necessary.

Two floating plants are required to perform all the bridge pier and abutment modifications in the Lower Granite and Little Goose Reservoirs. A single floating plant is required the following season to complete the bridge modifications in the Lower Monumental pool. No modifications are necessary in the Ice Harbor pool.

Following drawdown, final trimming of sheetpile and backfill with concrete can be done during low water periods. Some in-water work, consisting of diving and steel cutting is necessary during the late summer and fall time period. Concrete backfill will be placed within the cells via a concrete pump from the bridge deck or from the river bank.

W.3.5 Railroad and Highway Embankment Protection

Production of adequate quantities of rock, transportation of the rock, and placement of riprap for embankment protection and stabilization of drainage structures is one of the critical path elements in this implementation scheme. Two new quarries must be located and evaluated for suitable rock. Rock must be crushed into the proper sizes and barge transported to pre-determined underwater stockpile locations. This work must be completed for the respective reservoirs prior to drawdown of those reservoirs. Access to the work sites after drawdown may a difficult and time consuming process. Access over previously inundated rail and road beds may require significant measures to make viable. There is little latitude for adding contingency time during the barge transportation phase without extending the drawdowns one season later. Contingency time for placement of riprap will extend the placement season accordingly.

W.3.6 Drainage Structures Protection

For each reservoir, durations were determined and timeframes considered for all major functions necessary to load, transport, and place riprap materials for drainage structure modifications. Quantities derived for all treatments in each reservoir were added to the schedules. Durations for completion of the construction activities were then calculated based on selected productivity rates and numbers of crews to perform the tasks. Since this work will be done concurrently with reservoir embankment protection activities, appropriate work items have been included in the reservoir embankment protection schedules.

W.3.7 Railroad and Roadway Damage Repair

No schedule is provided for this task since it will depend on the nature and location of damage. Contract arrangement to perform the required repairs will be pre-negotiated so that contract forces can be mobilized to initiate repairs at the earliest possible time.

W.3.8 Lyons Ferry Hatchery Modification

The work required to modify Lyon's Ferry Hatchery will be done in two phases. The critical element is to modify the water supply pipeline prior to drawdown. The most expeditious method is to make modifications to the pipe pile system from a floating plant. Much of this pipeline is in the delta zone of the Palouse River. Access is difficult but not impossible. The installation of new pipe pile bents to stabilize the pipeline will require 6 months of in-water work.

The second critical element is to perform the necessary well modifications to restore the required hatchery water as soon as possible after drawdown. Wells will be modified or additional wells drilled to provide water lost by a drop in water surface. Pumps cannot be ordered until the wells are established and pump characteristics established. Up to 9 months lead time is required for pumps to be fabricated for this application.

During and immediately following the drawdown period temporary drain and overland flow piping is required until permanent structures can be constructed. Construction of those structures will be done during the late summer months after sufficient drainage of the bank deposits has occurred.

W.3.9 Habitat Management Units Modification

Habitat management unit modifications consist of revising and relocating irrigation system water intakes and installation of wildlife fencing. New intakes cannot be installed until after drawdown when the river location during low water conditions can be determined for each site. Intake structures will be prefabricated concrete units that can be placed on a prepared surface. The intake is mounted on the vertical face and the pump is mounted on the top of the unit. Fill material is placed out to the intake structure.

Temporary watering facilities are necessary for the summer and fall season following drawdown until the permanent intakes can be constructed. Temporary pumps mounted on trailers are one means to provide this interim water supply.

W.3.10 Reservoir Revegetation

Revegetation of exposed ground will commence within a few weeks after the start of drawdown of each reservoir. The schedule assumes an aerial application of seed and fertilizer on the exposed land mass for each reservoir on a 2-3 week interval.

W.3.11 Cattle Watering Facilities

Cattle watering facilities consist of a low capacity drilled well, a solar powered pump, and a ground level stock tank. Since the wells cannot be drilled until after drawdown, temporary watering facilities must be provided and maintained until the permanent system is complete. Temporary watering will be truck-hauled water to each watering site. Because access to some sites is difficult, installation of the temporary and permanent systems is estimated to take a long period of time.

W.3.12 Recreation Access Modification

Modifications to recreation areas are separated into two phases. Prior to drawdown, the critical feature is to establish an irrigation system for the areas to remain in operation. A combination of temporary and

permanent systems have been scheduled. Other modifications such as demolition of facilities, relocation of boat ramps, and construction of other facilities is scheduled to be completed following drawdown.

W.3.13 Cultural Resources Protection

Protection of cultural resources sites cannot proceed until the sites are exposed by drawdown. The construction schedules show that the work will commence immediately following drawdown and continue for a period of one year following drawdown.

W.3.14 Hydropower Facilities Decommissioning

The major activities include disposal of value items, disposal of hazardous wastes, and securing each project site. Site security facilities will be constructed as part of the site construction work. Concurrent work in securing the facilities to be abandoned will be done concurrently. Disposal of items of value can commence as early as January, immediately following drawdown. This work may require many months to complete. Identified hazardous wastes will be collected and disposed of concurrently with removal of items of value.

W.3.15 Non-Federal Implementation Tasks

Schedules were not developed for the following tasks. Those tasks were developed in order to provide a "ballpark" estimate of costs in order to make the appropriate economic evaluations. Those Non-Federal tasks are:

- Irrigation Systems Modification
- Water Well Modification
- Potlatch Corporation Water Intake Modification
- Other Water Intakes Modification
- Potlatch Corporation Effluent Diffuser Modification
- PG&E Gas Transmission Main Crossings Modification

W.3.16 Concrete Structures Removal

Schedules were not developed for this task. The purpose of providing a cost estimate for full removal of concrete structures is to provide an estimate of the cost of full removal. It is not a task that is part of the recommended activities to implement drawdown.

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Annex X

Comprehensive Baseline Cost Estimate

Annex X: Comprehensive Baseline Cost Estimate

X.1 General

The construction costs presented in this annex were developed by the Walla Walla District Cost Engineering Branch of the Corps of Engineers based on the scope of work, assumptions, and methodology presented in the companion engineering annexes (annexes A through V of this appendix). The following sections summarize specific details concerning the basis of costs for each of the engineering efforts and present cost summary tables for each effort. The comprehensive, detailed, cost estimates were developed using MCACES™ and are on file with the Walla Walla District Cost Engineering Branch of the Corps of Engineers (see Table X1).

X.2 Embankment Modifications

X.2.1 Cost

Components of construction include the following five cost elements: labor, permanent materials, construction equipment, subcontracts, and contractor's expendable supplies. The key ingredient in determining the cost of each of these elements is productivity of the work force and the construction equipment used to perform the various work activities. Productivity rates for the embankment excavation work were selected to reflect local weather, site conditions, work week hours, craft experience and availability, appropriate construction techniques, schedule sequencing, and experience gained on previous construction projects.

There is a difference between the cost for the riprap and the shot rock. The difference is attributable to two basic concepts: 1) riprap will be obtained from quarries, where the relative volume of useable material (yielding larger diameter riprap) is estimated to be about 40%; and 2) shot rock is more readily attained as quarries can generally produce a higher yield of rock that meet the requirements for size and gradation. These assumptions were made until further site-specific investigations, test blasting, test fills, and other fieldwork is performed.

Most costs were built up using databases for the cost of components of labor, materials, and equipment. In some cases, costs from the bid tabulations of previously bid and constructed projects were selected to represent the actual cost of similar type portions of this project (i.e., fabrication of trailers to haul fish). These historic values were then escalated to dollar values and adjusted for economies of scale and other factors to provide a rapid and relatively accurate reflection of the cost to do the work. A third source of prices included commercially available construction cost data guides. Generally, costs were built up for the most significant impact items, such as embankment dam excavation, driving interlocking steel sheet piles, levee construction, and production and transportation of riprap/shot rock.

Quantities were developed by Raytheon and are documented in the report entitled, *Embankment Excavation River Channelization and Removal of Concrete Structures*. Quantities developed in this report are intended to be "in-place" quantities. Factors such as swell and compaction are handled by adjusting the quantities in the estimate.

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY	TYPE OF COST	DESIGN & CONSTRUCTION DURATION	MID POINT OF CONSTR.		DAM REMOVAL OPTIONS DRAW DOWN CHANNEL BYPASS (Natural R Channel)	COMPLETE DAM REMOVAL (Natural River)
US Army Corps of Engineers® Walla Walla District — Cost Numbers are for Economic Study Purposes Only; Not Intended for Program Funding; Assumes Unrestricted Funds, No Escalation			OPTIONS ⟹		OPTION A-3a	OPTION A-3b
DESCRIPTIONS - for Ice Harbor, Lower Monumental, Little Goose & Lower Granite Locks & Dams -> Mc Nary Dam not included.					Thousand Dollars	Thousand Dollars
CONSTRUCTION AND ACQUISITION COSTS		Summary of Fish Improvements I & II			$858,939	$1,795,822
ANADROMOUS FISH EVALUATION PROGRAM		Annual Costs for 27 Years	Each Year	= =	$2,462	$2,329
ALL FOUR DAMS (Monitoring & Mitigation)				^ ^		
Anadromous Fish Evaluation Program Studies (AFEP)		27 Years	Each Year	^ ^	$2,462	$2,329
BREACHING DAMS		Summary of all the Breach Constr. Dams Costs Below			$858,939	$1,795,822
ICE HARBOR LOCK & DAM	Oct 98 Price Level	Summary		= =	**$206,902**	**$463,287**
Power House Turbine Modifications		2 Years	FY 2005	^ ^	$7,857	$7,875
Dam Embankment Removal		2 Year	FY 2005	^ ^	$65,524	$60,153
River Channelization		1 Year	FY 2006	^ ^	$35,349	N/A
Full Concrete Structure Removal		2 Year	FY 2007	^ ^	N/A	$298,016
Temporary Fish Handling Facilities		2 Year	FY 2005	^ ^	$19,702	$19,637
Project Dam Decommissioning		1 Year	FY 2006	^ ^	$1,477	$456
Railroad Relocations		2 Year	FY 2004	^ ^	$6,261	$6,299
Bridge Pier & Abutment Protection		3 Year	FY 2005	^ ^	N/A	N/A
Reservoir Embankment Protection		3 Year	FY 2004	^ ^	$44,892	$44,985
Drainage Structures Protection		3 Year	FY 2004	^ ^	$1,867	$1,872
Railroad and Roadway Damage Repair		3 Year	FY 2007	^ ^	$6,020	$6,032
Recreation Access Modification		2 Year	FY 2007	^ ^	$2,470	$2,472
HMU Modification		2 Year	FY 2006	^ ^	$3,238	$3,214
Reservoir Revegetation (For Air & Water Quality)		4 Year	FY 2007	^ ^	$8,237	$8,255
Cultural Resources Protection		2 Year	FY 2006	^ ^	$2,275	$2,281
Cattle Watering Facilities		2 Year	FY 2006	^ ^	$1,392	$1,399
Real Estate (Excessing Property)		4 Year	FY 2007	^ ^	$341	$341
LOWER MONUMENTAL LOCK & DAM	Oct 98 Price Level	Summary		= =	**$173,021**	**$415,590**
Power House Turbine Modifications		2 Year	FY 2005	^ ^	$7,857	$7,875
Dam Embankment Removal		2 Year	FY 2005	^ ^	$41,441	$39,663
River Channelization		1 Year	FY 2006	^ ^	$31,847	N/A
Full Concrete Structure Removal		2 Year	FY 2007	^ ^	N/A	$277,798
Temporary Fish Handling Facilities		2 Year	FY 2005	^ ^	N/A	N/A
Project Dam Decommissioning		1 Year	FY 2006	^ ^	$1,539	$472
Railroad Relocations		2 Year	FY 2004	^ ^	$13,921	$13,629
Bridge Pier & Abutment Protection		3 Year	FY 2005	^ ^	$6,414	$6,429
Reservoir Embankment Protection		3 Year	FY 2004	^ ^	$38,113	$37,789
Drainage Structures Protection		3 Year	FY 2004	^ ^	$2,062	$2,056
Railroad and Roadway Damage Repair		3 Year	FY 2007	^ ^	$4,753	$4,763
Recreation Access Modification		2 Year	FY 2007	^ ^	$2,043	$2,048
HMU Modification		2 Year	FY 2006	^ ^	$2,434	$2,439
Reservoir Revegetation (For Air & Water Quality)		4 Year	FY 2007	^ ^	$6,578	$6,593
Cultural Resources Protection		2 Year	FY 2006	^ ^	$1,578	$1,582
Cattle Watering Facilities		2 Year	FY 2006	^ ^	$2,459	$2,466
Lyons Ferry Hatchery Modifications		3 Year	FY 2005	^ ^	$9,710	$9,716
Real Estate (Excessing Property)		4 Year	FY 2007	^ ^	$272	$272
LITTLE GOOSE LOCK & DAM	Oct 98 Price Level	Summary		= =	**$192,134**	**$386,999**
Power House Turbine Modifications		2 Year	FY 2005	^ ^	$7,863	$7,863
Dam Embankment Removal		2 Year	FY 2005	^ ^	$26,589	$25,353
River Channelization		1 Year	FY 2006	^ ^	$53,462	N/A
Full Concrete Structure Removal		2 Year	FY 2007	^ ^	N/A	$250,941
Temporary Fish Handling Facilities		2 Year	FY 2005	^ ^	$18,052	$18,037
Project Dam Decommissioning		1 Year	FY 2006	^ ^	$1,471	$495
Railroad Relocations		2 Year	FY 2004	^ ^	N/A	N/A
Bridge Pier & Abutment Protection		3 Year	FY 2005	^ ^	$12,772	$12,772
Reservoir Embankment Protection		3 Year	FY 2004	^ ^	$39,718	$39,349
Drainage Structures Protection		3 Year	FY 2004	^ ^	$1,789	$1,781

TABLE XI

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY					DAM REMOVAL OPTIONS	
					DRAW DOWN	
US Army Corps of Engineers Walla Walla District		DESIGN &	MID		CHANNEL	COMPLETE
Cost Numbers are for Economic Study Purposes Only	TYPE OF	CONSTRUCTION	POINT OF		BYPASS	DAM REMOVAL
Not Intended for Program Funding	COST	DURATION	CONSTR.			
Assumes Unrestricted Funds, No Escalation					(Natural R Channel)	(Natural River)
			OPTIONS ====>		*OPTION A-3a*	*OPTION A-3b*
DESCRIPTIONS - for Ice Harbor, Lower Monumental, Little Goose & Lower Granite Locks & Dams -> Mc Nary Dam not included.					Thousand Dollars	Thousand Dollars
Railroad and Roadway Damage Repair		3 Year	FY 2007	^ ^	$9,814	$9,814
Recreation Access Modification		2 Year	FY 2007	^ ^	$3,257	$3,257
HMU Modification		2 Year	FY 2006	^ ^	$2,643	$2,633
Reservoir Revegetation (For Air & Water Quality)		4 Year	FY 2007	^ ^	$11,100	$11,100
Cultural Resources Protection		2 Year	FY 2006	^ ^	$1,435	$1,435
Cattle Watering Facilities		2 Year	FY 2006	^ ^	$1,973	$1,973
Real Estate (Excessing Property)		4 Year	FY 2007	^ ^	$196	$196
LOWER GRANITE LOCK & DAM	Oct 98 Price Level	Summary		= =	**$286,882**	**$529,946**
Power House Turbine Modifications		2 Year	FY 2005	^ ^	$8,130	$8,130
Dam Embankment Removal		2 Year	FY 2005	^ ^	$28,376	$26,231
River Channelization		1 Year	FY 2006	^ ^	$27,544	N/A
Full Concrete Structure Removal		2 Year	FY 2007	^ ^	N/A	$274,492
Temporary Fish Handling Facilities		2 Year	FY 2005	^ ^	N/A	N/A
Project Dam Decommissioning		1 Year	FY 2006	^ ^	$1,522	$487
Railroad Relocations		2 Year	FY 2004	^ ^	N/A	N/A
Bridge Pier & Abutment Protection		3 Year	FY 2005	^ ^	$32,672	$32,672
Reservoir Embankment Protection		3 Year	FY 2004	^ ^	$56,092	$55,395
Drainage Structures Protection		3 Year	FY 2004	^ ^	$2,838	$2,830
Railroad and Roadway Damage Repair		3 Year	FY 2007	^ ^	$109,420	$109,420
Recreation Access Modification		2 Year	FY 2007	^ ^	$7,973	$7,973
HMU Modification		2 Year	FY 2006	^ ^	$1,745	$1,746
Reservoir Revegetation (For Air & Water Quality)		4 Year	FY 2007	^ ^	$7,729	$7,729
Cultural Resources Protection		2 Year	FY 2006	^ ^	$1,538	$1,538
Cattle Watering Facilities		2 Year	FY 2006	^ ^	$1,037	$1,037
Real Estate (Excessing Property)		4 Year	FY 2007	^ ^	$266	$266
OPERATION & MAINTENANCE COSTS	Summary of Dam Routine & Minor Repair Costs, Each Year				$4,862	$4,862
ALL FOUR DAMS (Monitoring & Mitigation)	Oct 98 Price Level	Summary		= =	**$133,444**	**$133,444**
ANNUAL ROUTINE OPERATIONS, MAINTENANCE & REPAIR COSTS			Each Year		$4,633	$4,633
ICE HARBOR LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$1,631**	**$1,631**
LOWER MONUMENTAL LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$782**	**$782**
LITTLE GOOSE LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$630**	**$630**
LOWER GRANITE LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$1,590**	**$1,590**
MINOR - REPAIR COSTS	Annual Costs	Summary of the Dams	Each Year		$229	$229
ICE HARBOR LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$82**	**$82**
LOWER MONUMENTAL LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$39**	**$39**
LITTLE GOOSE LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$32**	**$32**
LOWER GRANITE LOCK & DAM	Annual Costs, Summary of Oper & Main Cost In the Detail Below,		Each Year		**$76**	**$76**
MAJOR - REPAIR & REHAB COSTS						
TURBINE UNITS & POWER HOUSE REHAB	Oct 98 Price Level	Summary, One Total Rehab only		= =	**N/A**	**N/A**
COSTS FOR OTHERS						
FISH HATCHERIES	Summary of Fish Hatcheries Operations, Minor & Rehab Costs		Each Year		$14,450	$14,450
FISH HATCHERIES OPERATIONS	Annual Costs	Summary	Each Year	= =	$13,762	$13,762
DWORSHAK FISH HATCHERY				^ ^	$2,250	$2,250
LOWER SNAKE RIVER FISH COMP PLAN				^ ^	$11,512	$11,512
INCLUDING WASHINGTON, OREGON, & IDAHO STATE ALSO NEX PERCE & CONFEDERATED TRIBES OF THE UMATILLLA						
FISH HATCHERIES MINOR & REHAB COSTS	Annual Costs	Summary	Each Year	= =	$688	$688
An assume costs that goes across the board.		5.0%				
BOR - WATER ACQUISITION AND TRANSACTION COSTS		Summary	Each Year	= =	$2,286	$2,286

LOWER SNAKE RIVER JUVENILE SALMON MIGRATION FEASIBILITY STUDY	TYPE OF COST	DESIGN & CONSTRUCTION DURATION	MID POINT OF CONSTR.	DAM REMOVAL OPTIONS DRAW DOWN CHANNEL BYPASS (Natural R Channel)	COMPLETE DAM REMOVAL (Natural River)
US Army Corps of Engineers® Walla Walla District — Cost Numbers are for Economic Study Purposes Only; Not Intended for Program Funding; Assumes Unrestricted Funds, No Escalation			OPTIONS ==>	OPTION A-3a	OPTION A-3b
DESCRIPTIONS - for Ice Harbor, Lower Monumental, Little Goose & Lower Granite Locks & Dams -> Mc Nary Dam not included.				Thousand Dollars	Thousand Dollars
AMOUNT OF THE WATER PURCHASED				427,000 Acre-Ft	427,000 Acre-Ft
PURCHASING WATER RIGHTS					
BOR - PURCHASING WATER RIGHTS for an extra 1,000,000 Acre-Ft		10 Years	Each Year		

The assumed swell factors are based on generally accepted values as follows:

- Impervious Core - damp; 1,990 Kg/m^3 (3,350 lb/cy); 67% swell; to 1,190 Kg/m^3 (2010 lb/cy);
- Earth rock Mixture - 25% E & 75% R, 31% swell,
- Gravel - wet, Good Gradation, 16% swell;
- Riprap Rock - Average; 2,670 Kg/m^3 (4,500 lb/cy); 72% swell; to 1,550 Kg/m^3 (2610 lb/cy).

Prevailing wage rates were obtained and payroll taxes and insurance applied as appropriate to wage and labor standards. The estimate uses Davis-Bacon Labor Rates from general decision WA980001, Modification 13. Materials prices were obtained from appropriate local supply sources, or estimated, based on the cost of erection and operation of site processing plants to handle large volumes of materials available at or near the site. Construction equipment rates for materials excavation, transportation and placement were established to include the cost of ownership, fuel consumption, maintenance and repair and other operations costs (except the labor for equipment operation). The source for these equipment rates is from *Construction Equipment Ownership and Operating Expense Schedule* EP 1110-1-8, Volume 8, September 1997.

Contractor's and subcontractors field office overhead, home office overhead and profit, were established using historical rates for similarly sized jobs and represent the contractor's cost of doing business and assuming the risks associated with construction work. The bond rates were also calculated.

X.2.2 Main Productivity Factor

For each of the construction scenarios, there is one key productivity factor, which controls the rate of material placement (or removal). The key productivity factor for embankment removal is the rate of excavation of the primary excavator. The productivity factor varies according to the amount of working space (related to the embankment elevation), the type and wetness of the material being excavated and the crew set-up needed to efficiently complement the selected types and numbers of primary excavators. The detailed elements of construction scheduling have not been optimized, but have been initially identified and used to set a pace of construction for the utilization and productivity of labor and equipment. Excavation of the earth embankment dam with impervious core could be economically performed with large hydraulic excavators and loaders at rates of 382 to 1,911 m^3/hr (500 to 2,500 cubic yards per hour) depending on the number of excavation units set up. Using a 6-day workweek with double shifting, embankment excavation and river channelization could be completed at all the dams by mid-January if drawdown begins on August 1. This pace combined with other activities, falls within the 8-month construction period for completion of the work.

X.2.3 Construction Equipment Selection

The type and size of hydraulic excavator selected for estimating this excavation was a CAT 5130 with a 10-m^3 (13-cy) bucket capable of producing 1,150 m^3 (1,500 cy) per hour. For cofferdam excavating and loading applications, a hydraulic excavator, with a rate of 320 m^3 (750 cy) per hour, was selected for material above the water surface and a dragline with a rate of 321 m^3 per hour (420 cy per hour) for material below the water surface. The material hauling units selected were CAT 777-c [82-metric ton (90-ton) capacity] end dump trucks for all zones. Haul distances from the borrow sites at the dams to spoil locations were scaled from the project area topographic maps.

Additional support equipment selected for placement and compaction of soil and rock materials included more conventional smaller-sized dozers, graders, track and rubber-tired backhoes, and water trucks. Performance rates for these equipment spreads were selected from manufacturer's handbooks and

adjusted by experience and site conditions. Costs were developed from *Construction Equipment Ownership and Operating Expense Schedule* EP 1110-1-8, Volume 8, September 1997 Additional costs were developed for drilling blasting, and processing costs, including sorting and crushing, of blasted rock. A barge and tug are part of the floating plant used for underwater drilling, blasting, and excavation.

X.3 Bridge Pier Modifications

The construction cost of modifications to the bridge piers and abutments for the Lower Snake River reservoirs were estimated based on site-specific data discussed in Section 3 of the *Lower Snake River Reservoir Stabilization Plan* (Raytheon 1997). The estimate assumes that required riprap will be placed from barges prior to drawdown. The sheetpile will also be driven from floating plant. Once drawdown has occurred final dressing of the riprap will occur in the dry.

X.4 Reservoir Embankment Modifications

The construction cost of embankment protection for the Lower Snake River reservoirs was estimated based on quantities developed from information obtained from contracts let for relocation of the railroads and aerial photographs taken prior to filling of the reservoirs. Quantity takeoffs for these protection measures were based on dimensions developed by the Walla Walla District Engineering Division. Quantities were calculated separately for each embankment segment on each of the four reservoirs. A cost was developed for production of riprap based on crews required for drilling and blasting, assumed overburden depth, drill pattern, powder factor, yield of material, secondary blasting, handling of material, sorting and crushing. The other component of the proposed riprap protection was the cost of barge transportation and stockpiling in three of the reservoirs prior to drawdown and hauling from the stockpiles and quarries and placement at the site with final dressing of the slopes after drawdown of the reservoirs occur.

X.5 Reservoir Drainage Structure Modifications

The construction cost of drainage modifications for the Lower Snake River reservoirs were estimated based on site-specific data and generic sketches and layouts of modifications discussed in Section 6.3 of the *Lower Snake River Reservoir Stabilization Plan* (Raytheon 1997). Quantity takeoffs for these modifications were based on dimensions shown on plan and section drawings for the proposed modifications (see Plates 6-9 to 6-12) and site-specific elevations and slope distances for all identified drains. Quantities were calculated separately for each drain location and combined into an estimate of the cost to construct all drain modifications on each of the four reservoirs.

The total costs for riprap blanket slope protection, riprap blankets for energy dissipation, cleaning of exposed and submerged culverts, additional new culverts, and new combined drainage flow culverts in each of the four Lower Snake River reservoirs was then estimated. Slope protection treatment details and quantity worksheets for each reservoir are shown in the Raytheon Report (Raytheon 1997).

Horizontal borings were estimated based upon available data for large diameter casings. A large portion of the total cost is involved in mobilizing and setting up the boring pit, aligning guiderails for the boring machine, and machine assembly. It was assumed that areas of horizontal borings would be accessible by existing roads.

The number of contract packages to execute the reservoir drainage modification work is assumed to be two contracts, one for riprap material supply and a separate one for installation. As two reservoirs are to be worked concurrently, this is probably the optimum arrangement for contract administration.

X.6 Road and Railroad Repair Plan

There are approximately 68 potential failures that may occur. This assumed number is based on problem areas observed during the 1992 drawdown. The total embankment repair cost could vary significantly from the present estimate. Some embankment failures may occur in areas that were not identified by this study; however, it is also expected that some of the areas identified for potential failure will not fail. Because of these uncertainties a relatively high contingency was used.

X.7 HMU Modifications

There are eight HMUs with a total of 11 surface water intake pump stations. An average increased pump requirement and piping distance was determined and used as a basis for developing the total cost for modifying all 11 pump stations. The following criteria were used to develop the cost estimate:

1. All new piping will be 300 millimeters diameter
2. The average distance of the piping will be 300 meters
3. The average water requirement will be 79 liters/second
4. The average pump size will be 100 horsepower
5. The local power company will supply power, but the USACE will pay for trenching

The two HMUs that use a well-supplied water source will also require significant modifications. It is assumed two new wells will have to be drilled and, at a minimum, require 92 meters of additional drilling below the existing wells depths to maintain the water supply. With this additional depth, higher horsepower pumps will also be required. The estimate also provides for temporary water supply to the existing system via a trailer mounted pump system that could be moved as the water level recedes.

X.8 Cultural Resources Protection Plan

All activities described below will be carried out in compliance with applicable cultural resources laws and regulations. This includes coordinating and consulting with the appropriate State Historic Preservation Office, Tribe(s), and other interested parties.

Mobilization/demobilization costs were factored based on the mileage from either Pasco, WA, or Lewiston, ID, to each reservoir group for sites determined to be accessible by highway, railroad, or currently submerged roadway. Mobilization/demobilization costs for the remote sites were estimated assuming access either by helicopter or boat. Assumptions for remote sites were that equipment, personnel, and material would be trucked to a staging area. From there they would complete the trip to the site via boat or helicopter. It was assumed that 10% of the sites would be accessed by boat while 5% would be accessed by helicopter.

The complement of equipment used for the bulk of site protection consists of an 8 m^3 (10 cubic yard) dump truck, pulling a flatbed tilting trailer, with a small front-end loader, and a crew/miscellaneous tool truck. The work crew consists of 4 individuals, 1 loader operator, 1 truck driver, 1 laborer, and 1 working supervisor. Labor tasks will be performed by all crew members. During work activities at remote sites, either a boat and trailer or helicopter will be added.

Since site locations are not specifically identified and each site is relatively small, it was assumed that equipment would be mobilized to each site each working day. Maximum and minimum mileage was computed to sites in each reservoir from the closer of Pasco, WA or Lewiston, ID. The average distance to each reservoir was then used to calculate travel time for the crew and equipment.

The operations required to protect the cultural resource sites include:

- Grading and preparing the site including leveling the site as necessary and manually preparing the surface and placement and securing the geomembrane.
- Placing and compacting a 0.3-meter layer of random fill material. The fill material will be borrowed from any convenient nearby location.
- Preparing the seed bed (manually), applying seed (manually), and placing and securing the erosion protection material for the re-vegetation process.
- Pre-place riprap, gravel, and highway base materials (assumed) during the bank protection operations. The costs are the same as those developed for production and transportation of such materials. The total costs are based on calculated volumes for each type of site.

Access to remote sites by boat or helicopter is estimated by adding this type of equipment to the crews and substituting a bobcat for the small front-end loader.

X.9 Project Decommissioning Plan

X.9.1 Mothball Option Cost Estimate

To develop the mothball option cost estimate for all four Snake River dams the different equipment and project features at the other three dams were evaluated and compared to Lower Granite Dam. The major identified differences that would increase the effort and cost for decommissioning are as follows:

- Little Goose fish pumps are turbine driven whereas Lower Granite fish pumps are electrically driven. Turbine driven pumps will require additional labor to prepare them for long extended outages. Corrosion of the unit would be a major concern
- Lower Monumental downstream and upstream navigation lock gates at are lift gates whereas the navigation lock gates at Lower Granite are miter gate and tainter gate. Lift gates require the use of cables in their operation and their storage will need to be considered if long term outages are expected. In order to prepare the cables for storage, the load on the lifting cables will have to be lessened by the blocking of the counter weights. Once this is done, the cables can be removed for storage in the powerhouse. This is necessary in order to prevent deterioration of the cables over a long-term outage. The added labor will cause a substantial increase in the cost of decommissioning the Dam. This cost is based on a recent contract at Ice Harbor Lock and Dam where the downstream lift gate was removed and replaced.
- Ice Harbor Lock and Dam has eight fish pumps on the south shore rated at 250 horsepower and three on the north shore rated at 200 horsepower. Lower Granite has two fish pumps at 800 horsepower each. There will be an increase in the cost due to this difference.
- The downstream gate at Ice Harbor is a lift gate with the upstream gate being a tainter gate. The gates at Lower Granite are miter gates. The lift gate require the use of cables in their operation and their storage will need to be considered if long term outages are expected. In order to prepare the cables for storage, the load on the lifting cables will have to be lessened by the blocking of the counter weights. Once this is done, the cables can be removed for storage in the powerhouse. This is necessary in order to prevent deterioration of he cables over a long-term outage. The added labor will cause a substantial increase in the cost compared to that of Lower Granite.

The cost for the Mothball Option will have a one-time cost component and an annual cost component. The total cost to mothball will depend on the number of years the dams are inactive.

X.9.2 Abandon Option Cost Estimate

The abandon option consists of costs to secure the four sites. This is done by placing a fence around the area and securing/hardening all openings.

X.9.3 Cost Estimate for Hazardous Materials

The estimated cost for disposal of hazardous materials, substances, chemicals and wastes at all four projects was estimated by obtaining an inventory to develop the quantities. A crew was developed to assemble the wastes at an on-site collection area. Costs for disposal were based on the current District hazardous waste removal contract.

X.9.4 Project Security Cost Estimate

Costs for continued security were not included in the construction cost estimate, however they are presented here. The annual cost shown for surveillance is based on one person inspecting a project one time per month. Table X2 shows the estimated cost for project security.

Table X2. Estimated Cost For Security

Item	Annual Cost
Manned surveillance	$5,000
Total Cost For Lower Granite Dam	$5,000
Total For Little Goose Lock and Dam	$5,000
Total For Lower Monumental Lock and Dam	$5,000
Total For Ice Harbor Lock and Dam	$5,000
Total Cost For All Four Snake River Dams	**$20,000**

X.9.5 Total Cost Estimate for Recommended Decommissioning Option

The abandon option is recommended for the four Snake River dams. The items included in this option are:

- Install facilities to backfeed power into the project from the existing grid so the existing lighting system can be used.
- Weld Navlock and spillway gates shut.
- Install security fences and signs.
- Secure and harden entrances to structures.
- Dispose and treat hazardous waste.

It is assumed that excess equipment and property will be sold off. Any funds received will offset the cost of removal and transportation.

X.10 Contingency Analysis

The goal in contingency development is to identify the uncertainty associated with an item of work or task, forecast the risk/cost relationship, and assign a value to this task that will limit the cost risk to an acceptable degree of confidence.

Contingencies were developed at a meeting held on August 18, 1998, with knowledgeable project personnel. Each task was analyzed and contingencies were developed based on the risk factors and uncertainties involved. An overall contingency was developed by applying these contingencies to the direct costs of the tasks and obtaining a weighted average.

Contingency guidance is provided in ER 1110-2-1302. For a reconnaissance/feasibility level, contingencies of 20 percent are considered reasonable for projects over $10 million and contingencies of 25 percent for projects less than $10 million. These overall contingency factors are a guide for contingency development and are not intended to restrict or limit contingencies to these values. Table X3 shows the contingencies assigned and the reasoning for the determinations.

Table X3. Contingency Analysis for Levee/Channelization Option

Task Description	Contingency Percentage	Reason for Assigned Contingency
Powerhouse Turbine Modifications	30%	Uncertainty regarding the routing of plumbing for cooling modifications and what additional controls and instrumentation would be required.
Dam Embankment Removal	20%	Feasibility-level-of-detail risks involved in moving large amount of material in short time while reservoirs are being drawn down. Quantities and procedures fairly well defined.
River Channelization	30%	Final alignment and quantities involved are uncertain. Model studies and bathymetric surveys are required.
Temporary Fish Handling Facilities	30%	High uncertainty in number of fish to be hauled.
Project Decommissioning	40%	Uncertainty in quantities of waste to be disposed of and requirements to harden structure to keep trespassers out.
Railroad Relocations	30%	High uncertainty as to requirements railroads will impose on new track alignment.
Bridge Pier & Abutment Protection	25%	Uncertainty in quantities and ability to perform installations under bridge structure.
Railroad and Highway Embankment Protection	35%	Uncertainty in viability of existing access roads to accommodate construction traffic. Access and slope conditions not full defined
Drainage Structures Protection	40%	Access to drainage structures is very problematic and high uncertainty because many drainage structures are located beyond the limits of embankment protection activity.
Railroad And Roadway Damage Repair	75%	Extremely high uncertainty as to extent of damage that will be caused by rapid drawdown of reservoirs. Amount of damage could easily double.
Recreation Access Modification	20%	Fairly well defined quantities and standard procedures contingency below average for feasibility level.
Lyons Ferry Hatchery Modification	30%	Uncertainty in depth to which wells will have to be drilled in order to obtain water after drawdown. Unknown condition of long-term sediment accumulation around pipeline. Will dredging be required in order for floating plant to have access to perform work?
Habitat Management Unit (HMU) Modifications	20%	Generally good idea of what is required to modify HMUs uncertainty exists in sizing of pumps and requirements of where to place intake structures after drawdown.

Table X3 continued. Contingency Analysis for Levee/Channelization Option

Task Description	Contingency Percentage	Reason for Assigned Contingency
Reservoir Revegetation	30%	Risk involved in aerial operations that are dependent on weather (i.e., high winds in canyons); also uncertainty in the extent of replanting that would be required. The success rate of aerial seeding is also suspect.
Cultural Resources Protection	100%	Uncertainty in site quantity, location, and access: since no vegetation would remain after drawdown, it is extremely likely that new sites would be discovered.
Cattle Watering Facilities	30%	Uncertainty in location and depth of wells.
Total		
Weighted Average Contingency	34%	